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APPLICATIONS OF THE NEW AUSTRIAN TUNNELLING METHOD IN URBAN AREAS

University — Université

UNIVERSITY OF ALBERTA

Degree for which thesis was presented — Grade pour lequel cette thèse fut présentée

MASTER OF SCIENCE

Year this degree conferred — Année d'obtention de ce grade

1984

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APPLICATIONS OF THE NEW AUSTRIAN TUNNELLING METHOD IN URBAN AREAS

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A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA
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DEDICATION

To Uschi and Ju

ABSTRACT

The behaviour of soft ground urban tunnels excavated using the New Austrian Tunnelling Method (NATM) is the subject of this research. Essential features of the method are outlined. Attention is also given to applications of the NATM to underground excavations with large cross-sections, used for double track subway tunnels and subway stations.

Field measurements of ground deformations caused by NATM tunnels are analysed according to indices commonly used in tunnel engineering practice. Conclusions regarding possible limitations of the method are derived. Requirements for limiting the extent of ground movements around NATM tunnels are outlined.

Two- and three-dimensional numerical analyses using the Finite Element Method have been used to model an NATM case history and to study the stress-displacement behaviour close to the face of shallow tunnels. The results indicate that despite the markedly three-dimensional behaviour close to the tunnel face, the final equilibrium condition in terms of stresses and displacements can be more economically obtained using simplified two-dimensional techniques.

ACKNOWLEDGEMENTS

This research was supervised by Dr. Z. Eisenstein, to whom I am deeply indebted for his continued encouragement and advice. Appreciation is extended to Drs. P.K. Kaiser and D.W. Murray, who have always found time for discussing various aspects of the project. Thanks are also due to Dr. J.G. MacGregor for his meticulous review of the text.

The work reported herein has benefitted greatly from discussions with my many graduate colleagues. A very sincere thanks goes to Arsenio Negro, who made a great deal of valuable contributions at critical stages. Special thanks go to Paulo Branco and Patrick Collins, who critically reviewed substantial portions of the manuscript. Pat's helpful computer programs and sense of humour made those many nights spent in the computer room a little more enjoyable. José Brandt, Dave Chan, Marv Cherniawski and Ivan Corbett also helped me in different ways in the course of this research work.

Assistance with the computing work was provided by Mr. T.A. Casey on many occasions. Dr. E. Evgin provided useful information in the initial stages. This does not go unrecognized.

Material for parts of this research was obtained through personal correspondence with several individuals and companies. Some of them must be singled out for their extraordinary interest and assistance, including Dr. U. Arslan, Dr. E. Gartung, Mr. J.G. Gehring, Prof. H. Duddeck,

Dr. F. Massad, Prof. L. Müller, Dr. D. Stroh and Interfels GmbH.

Financial assistance provided by scholarships from the Rotary Foundation, from the Instituto de Pesquisas Tecnológicas do Estado de São Paulo and later by the City of Edmonton is gratefully acknowledged. I also wish to thank the Department of Civil Engineering of the University of Alberta for awarding me the E.B.A. bursary and for providing computer funds for the project.

In closing, a very heartfelt thanks must go to all members of my family. To my parents Dirce and Heinrich, for their continued interest in my education. To my daughter Juliana, who made the busiest moments a lot happier. To my wife Ursula, I wish to express my indebtedness for her help in the typing of the manuscript and in the numerous tasks that are involved in the preparation of a thesis. The completion of this work would not have been possible without her love and encouragement. Besides helping me, she still managed to be an excellent mother and a first class graduate student. I am glad we made it together!

"We are condemned to civilization.
Either we progress or we disappear."

Euclydes da Cunha
'Os Sertões' (1902)

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1. INTRODUCTION

1.1 General

The last two decades have witnessed several major accomplishments in the art of tunnelling in soft ground. Modern tunnel boring machines and the New Austrian Tunnelling Method (NATM)[1], associated with technologies such as shotcrete, make it possible to excavate any type of soft ground tunnel in heavily populated urban areas in a safe and economical manner.

Moreover, the development of computational techniques such as the Finite Element Method (FEM) has allowed a better understanding of the processes involved during tunnel excavation. Theoretical analyses which use tools such as the FEM, in combination with field observations of deformations and loads, allow recognition and interpretation of the complex tunnel excavation mechanisms and consequent establishment of appropriate design criteria.

1.2 Objectives and Scope of this Thesis

Although some general considerations about the NATM are presented in the following sections, the present work is mainly aimed at analysing the NATM from a geotechnical perspective. Attention is concentrated on identifying the construction factors that affect the ground movements around a tunnel and on the development of radial stresses and displacements close to the tunnel face.

---

[1] The NATM is sometimes refered to as the 'shotcrete method'.

Chapter 2 attempts to unify field results of deformations around NATM excavated tunnels according to indices commonly used in tunnel engineering practice. Conclusions regarding design considerations of practical importance are derived.

Chapter 3 is concerned with special applications of the method to urban excavations with large cross-sections, used for double-track subway tunnels and subway stations. A classification of large cross-section excavation schemes commonly found in practice is presented. Selection criteria for the excavation scheme are outlined.

Chapters 4 and 5 are aimed at understanding some of the mechanisms generating displacements and pressures during excavation of shallow tunnels by the NATM. To address these problems, it has been necessary to employ two and three-dimensional analyses using the Finite Element Method (FEM). Back analyses of a case history by two and three-dimensional FEM allows verification of the adequacy of these techniques in analyses of NATM tunnels.

Chapter 6 summarizes the findings of this research project, emphasis being placed on practical implications on current design procedures. Recommendations for future research are outlined.

Detailed descriptions of several case histories, mostly derived from German literature, are presented in Appendix A, with the intent of providing illustration for several parts of this thesis. Design concepts of practical significance are outlined in Appendix C.

1.3 Need for "Non-Shield" Tunnelling Methods

Nowadays, tunnelling in soft ground in North American practice is mostly carried out with the aid of tunnel boring machines which are frequently termed 'shields' or 'moles'. The necessity for other procedures has been recognized by Peck et al. (1972:260):

> 'Today, almost all circular and some horseshoe tunnels are excavated within the protection of shields... This aspect of the state of soft-ground tunneling is unfortunate because complex modern underground systems involve many geometrical forms not adaptable to shield tunneling. These include junctions of rapid transit lines, stations formed by excavating between shield driven tubes, escalator and stairway passageways and their junction with driven tunnels, and a host of auxiliary structures for various purposes.'

These authors further point that due to mechanization of excavation today's tunnellers have lost much of their skill in hand mining:

> 'The lore of hand mining in unfavourable ground is almost forgotten; men with a variety of experience in directing hand mining are a vanishing breed; skilled, soft-ground hand miners are rare.

> Unsatisfactory and inept methods, discarded and replaced by better ones many years ago, are being revived through ignorance of the lessons of the past. It is a matter of concern to all interested in soft–ground tunneling that our emphasis on progress and mechanization is causing us to lose an important and useful heritage.'

These words enhance the necessity for tunnelling procedures such as the New Austrian Tunnelling Method, outlined in the following sections. In this method, the workman is put in direct contact with the ground and frequently forced to make decisions based on experience. It may also be inferred that there is an implicit requirement that experienced tunnellers be involved in non-shield tunnelling.

1.4 The New Austrian Tunnelling Method (NATM)

The term 'New Austrian Tunnelling Method' has apparently been introduced by Rabcewicz (1963). The origin of this terminology is related to the existence of tunnelling methods used in olden times to which several national names were applied[2]. The NATM is a procedure for excavation and support of tunnels which is adaptable to ground conditions varying from hard rock to soft ground. Support is provided by suitable combinations of shotcrete, steel ribs, welded wire mesh and anchors.

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[2] Most of these old methods are described in detail by Drinker (1878).

Excavation of a tunnel in soft-ground by the NATM frequently makes use of a 'heading and bench' procedure, as shown in Figure 1.1. The support is erected immediately after excavation and closed to a full ring as close as possible to the tunnel face. Plate 1.1 shows a detail of installation of the steel ribs close to the face.

It is important to point out that the NATM has been used for 'regular cross-section' tunnels (i.e., maximum of 30 to 40m² of cross-sectional area) and for 'large cross-section' tunnels (i.e., more than 60m²) of cross-sectional area). Composite sections have also been used in the construction of underground subway stations and junctions of subway lines, as described by Laue et al. (1978) or Petruschke (1980). In the double-track tunnels and stations, multiple staged excavation techniques are used (see Chapter 3), but the heading and bench technique is normally kept as a way to minimize displacements and maintain face stability within each stage. Also, NATM tunnels with small dimensions are frequently hand mined.

The NATM is frequently presented as a 'philosophy' instead of a tunnelling method, which includes design, construction and performance monitoring in an integrated manner (e.g. Müller and Fecker, 1978). In this thesis, the term NATM is used to define any tunnel constructed with 'non-shield' techniques and shotcrete linings, an in-depth assessment of this 'philosophy' not being attempted. However, it is important to highlight some essential

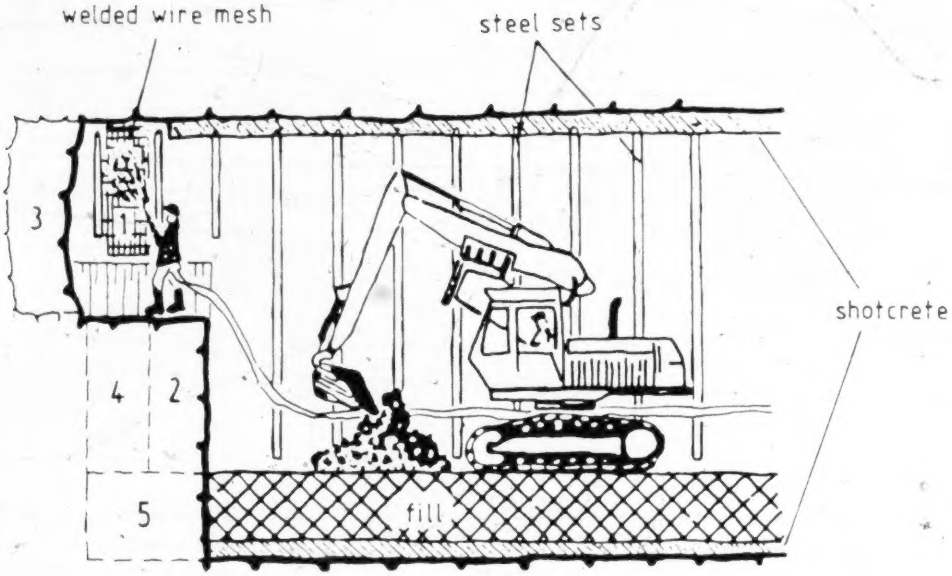
welded wire mesh
steel sets
shotcrete
fill
1
2
3
4
5

Figure 1.1 Typical scheme for excavation of NATM in soft-ground (after Pierau, 1982:modified)

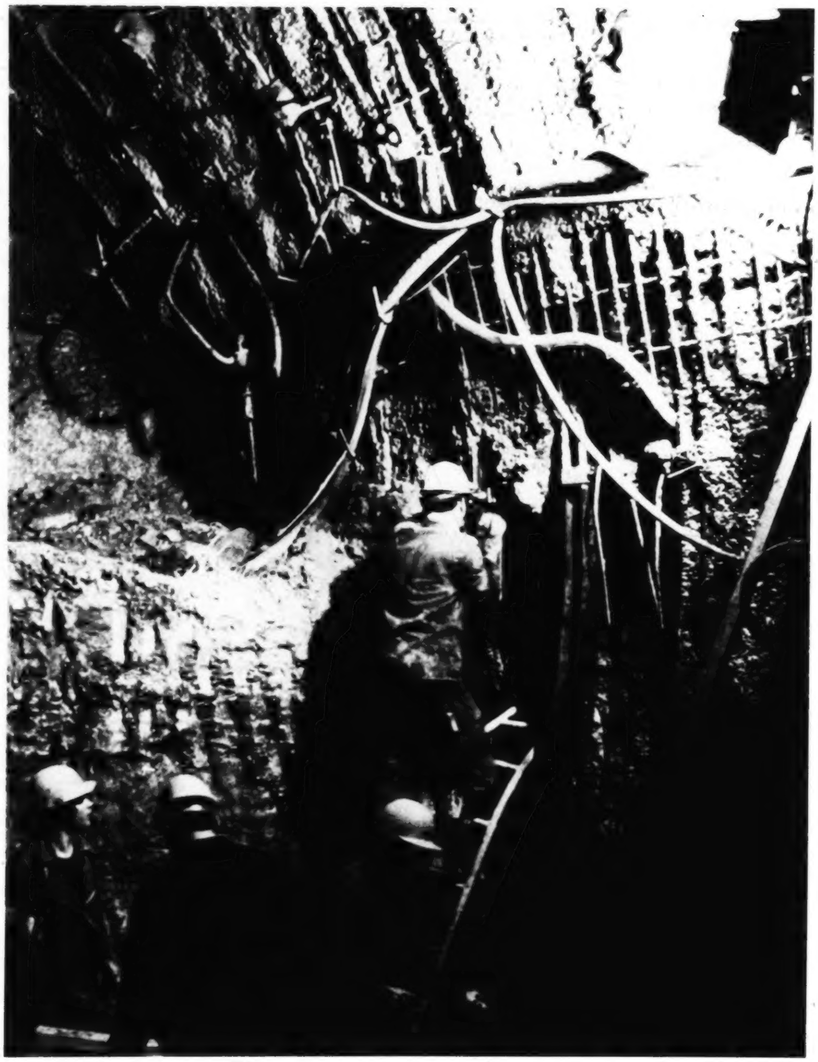

Plate 1.1 Installation of steel sets close to the tunnel face (from 'U-Bahn Linie 8.1', reproduced with permission of the U-Bahn Referat - München)

features, specially those that differentiate the NATM in soft ground from that in hard rock. This is done in the following sections.

1.4.1 Rock vs. Soft Ground NATM

Most of the early applications of the NATM were to tunnels constructed in rocks subjected to high in-situ stresses (Brown, 1981:15). In these cases, surface movements were generally of little concern and deformations were frequently allowed in order to achieve a 'pressure relief' and therefore decrease the amount of necessary support.

In shallow urban tunnels the in-situ stresses are normally much smaller and the requirement for keeping the ground movements minimal overrides the problem of minimizing support quantities. The need to properly understand the differences between both cases has been recognized by authors such as Rabcewicz and Pacher (1975:320) and Brown (1981:16).

This distinction is best understood from an analysis of ground-support interaction diagrams, a tool which was frequently used by Rabcewicz and his co-workers in order to explain the NATM concepts. In these diagrams, the ground and support behaviour are represented by its 'characteristic curves'. Figure 1.2 is an attempt to illustrate this concept, which is used in the Convergence Confinement Method (e.g. Brown et al., 1983). For simplicity, it is assumed that a deep circular tunnel is being excavated in a ground mass subjected to a hydrostatic stress field. This problem is totally axisymmetric thus the stresses and displacements in the ground around the tunnel and in the support elements will not vary along the tunnel perimeter.

As tunnel excavation progresses, the originally existing radial support pressure (P_0 in Figure 1.2b) is gradually removed, stresses being redistributed part to the ground and part to the support. As this support pressure is reduced, radial deformations (u) will occur along the tunnel perimeter and the pressure-displacement behaviour will follow the 'ground characteristic curve' or 'ground response curve' depicted in Figure 1.2b.

Figure 1.2c shows a ground characteristic curve and two 'support characteristic curves' or 'support reaction lines' for two support systems having the same load-deformation properties but installed at different distances from the tunnel face. These support elements will deform and attract load as the ground supporting pressure is reduced by excavation of the tunnel.

In deep rock tunnels it is desirable to mobilize the strength of the rock to a certain extent, minimizing the loads which will be carried by the support. In this case, the support represented by line (2) is appropriate. Some delay in erecting this support is allowed and radial displacements u_d will occur.

In shallow soft ground tunnels the idea of allowing a 'pressure relief' is generally not applicable because this

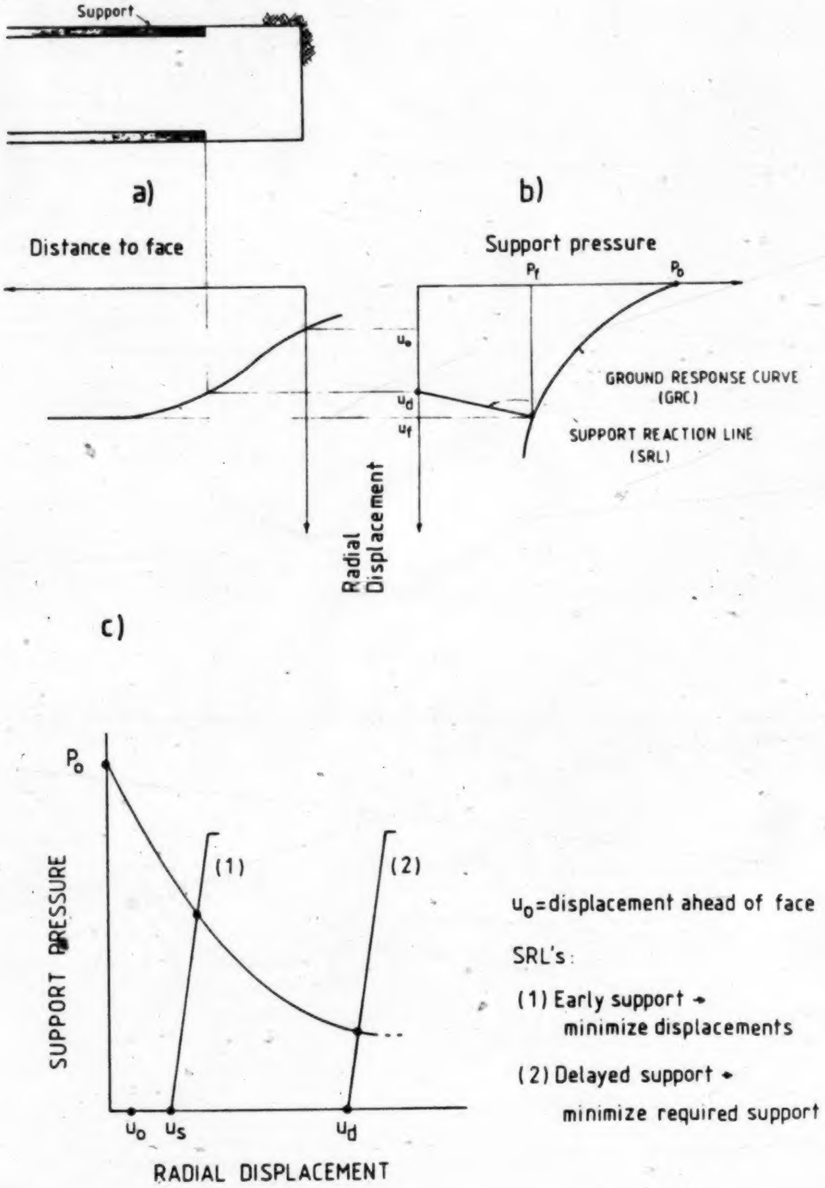
Support
a)
b)
Distance to face
Support pressure
P_f
P_o
u_e
u_d
u_f
GROUND RESPONSE CURVE (GRC)
SUPPORT REACTION LINE (SRL)
Radial Displacement
c)
P_o
SUPPORT PRESSURE
(1)
(2)
u_o
u_s
u_d
RADIAL DISPLACEMENT
u_o=displacement ahead of face
SRL's:
(1) Early support → minimize displacements
(2) Delayed support → minimize required support

Figure 1.2 Ground–support interaction diagrams

will necessarily increase the displacements around the tunnel and at the surface. However, by the time the support is erected, some radial displacement u_s has already been experienced, with a correspondent reduction in pressure. In order to minimize settlements, it is important that this displacement u_s be as close as possible to the unavoidable displacements that occur ahead of the tunnel face (u_0).

1.4.2 Shotcrete Lining

The shotcrete lining is an essential feature of the NATM in soft ground. Besides the possibility of providing support immediately at the tunnel face, the shotcrete lining fills the voids at the ground-lining interface in a very effective manner. These characteristics contribute to hinder ground deformations, a vital issue when tunnelling in urban areas. In soft-ground, a typical support system utilized in a single-track subway tunnel is comprised of:

1. Welded wire mesh (full circle).
2. Shotcrete applied in layers up to 25cm.
3. Light weight steel ribs spaced 0.8 to 1.2m (permanent, full circle).
4. Anchors[3].

Shotcrete has been traditionally used as a temporary support. In recent years, however, a trend to use shotcrete as permanent support in soft ground tunnels is noticeable (e.g. Eckschmidt and Celestino, 1982). In these cases, the support is normally augmented in two or more stages at increasing distances from the advancing face and the usual distinction between 'temporary' and 'permanent' support is not applicable.

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[3] The effectiveness of anchors as a support aid in soft ground is frequently questioned (e.g. Schulz and Edeling, 1972; Laabmayr and Weber, 1978). It appears that in soft ground the use of short anchors is related to the problem of holding the steel sets in place during construction operations (Steiner et al. 1980:321).

A comprehensive review of the role of shotcrete, its properties and structural behaviour of the shotcrete lining will not be presented in this thesis. Details of specific applications in soft ground tunnels can be found in most of the papers referenced in Appendix B. Other extensive publications of interest are the proceedings of the ASCE–ACI Conferences on Shotcrete for Underground Support (ASCE, 1974; ASCE, 1977) and some reports of the U.S. National Technical Information Service – NTIS (e.g. Mahar et al., 1975; Brekke et al., 1976). Innovative approaches like the use of fiber reinforced shotcrete or applications under compressed air conditions are reported by Henager (1981) and Distelmeier (1981) respectively.

1.4.3 Field Instrumentation

Use of monitoring devices during tunnel excavation is a feature of the NATM which has been emphasised since its earliest applications reported for example by Rabcewicz (1964/1965). In soft ground, instrumentation is used routinely to check on the performance and safety of the tunnel. Potential exists for application of an 'observational method of design', where monitoring would serve to indicate possible design changes as excavation progresses. Measurements in the temporary support may also be used in the dimensioning of the definitive one (Brown, 1981:17).

Regarding details of instruments and instrumentation layouts, it is verified that no substantial difference exists between NATM arrangements and those which have been used in shield projects. A feature found in many of the NATM cases examined and not commonly found in shield projects is the use of hydraulic pressure cells of the Glötzl type. These cells are frequently used for the measurement of stresses in the shotcrete lining and between the lining and the ground. Although no specific review will be carried out herein, it is important to point out that reliability of these instruments is highly dependent on the installation procedure. Important observations in this regard were published by Brekke et al. (1976), Sauer and Sharma (1977) and Gesta et al. (1980).

1.4.4 Contracts and Costs

Application of the NATM as envisaged by Rabcewicz and his co-workers requires a high level of understanding between owner, designers and contractors. Brown (1981:17) observed:

'All parties involved in the design and execution of

> project – design and supervisory engineers and the contractor's engineers and foremen – must understand and accept the NATM approach and adopt a co-operative attitude to decision making and the resolution of problems.'

European and specially Austrian contractual practice have developed some characteristics that facilitate the execution of tunnels by the NATM. Some of these characteristics are discussed by Muir Wood and Sauer (1981) and Steiner et al. (1980). The latter publication is quite extensive and was aimed at examining the reasons why European construction practice is often less expensive and involves less litigation than that in North America.

Another major issue in urban tunnelling are the costs involved. It is often pointed out that since the cost is a direct function of the rate of advance the NATM, which generally yields slower excavation rates, will necessarily be a more expensive technique than the traditional shield. Although the NATM is normally a slower method than the shield, the observation above is misleading and it has been observed that the NATM can result in considerable savings on the total costs (e.g. Clough, 1981:157; Smith, 1984:21). Other comparisons between costs of NATM and shield tunnels are presented by Klawa and Schreyer (1979).

Subway construction in the city of Munich is an example of how the NATM became competitive with the shield method in terms of global costs. This is illustrated in Figure 1.3, which also shows that a period of approximately two years from the initial application was required until the new method could effectively be established as a cost-saving procedure[4].

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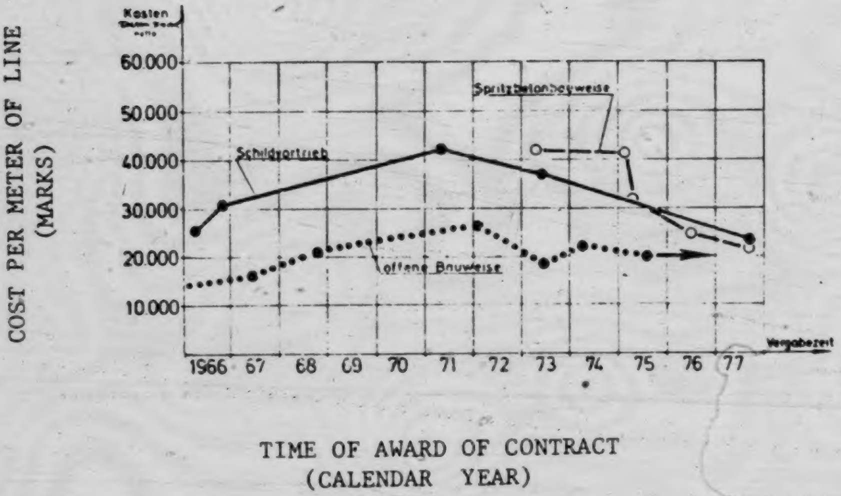
[4] Other advantages of the NATM over the shield are reported by Steiner et al. (1980:118).



Spritzbetonbauweise:	Shotcrete Support (NATM)
Schildvortrieb:	Shield Tunnel
Offene Bauweise:	Open-cut Construction

Figure 1.3 Variation of tunnel construction costs with time in the city of Munich (after Hochmut, 1978:modified)

2. OVERVIEW OF NATM GROUND CONTROL

2.1 Introduction

Most soft ground tunnels in North America are currently excavated by the shield method. In order to introduce a new procedure like the NATM in regions where sound shield tunnelling records exist, one has to consider contractual, economical, operational and technical factors. Only the technical aspect 'geotechnical performance' will be analysed herein, with basis on the review of several case histories. To assess aspects other than the geotechnical performance, a review of the publication by Steiner et al. (1980) is recommended.

In order to evaluate the geotechnical performance of the NATM cases reviewed, it is convenient to classify these cases according to performance indicators commonly used in engineering practice. Section 2.2 offers a brief review of some of these indices. An attempt to classify the NATM cases according to them follows (Section 2.3).

The material presented in Section 2.4 is intended to provide a brief insight into the mode of ground deformation around tunnels excavated by the NATM. This is done based on empirical evidence from case histories. Special emphasis is placed on identifying the main sources of soil displacements. Section 2.5 is included as an effort to shed some light on the problem of how to assess limitations of the NATM when applied in soft ground tunnels.

2.2 Geotechnical Performance Indicators

The following sections offer a brief review of some indices traditionally used in tunnel enginnering practice. Some of the concepts presented assume an oversimplified ground behaviour during tunnel excavation, such as for example an ideal undrained soil response. However, the lack of objective theories which could take into account the numerous variables in an actual tunnelling problem still favours the use of simplified procedures, generally of undoubted value to the engineer engaged in day-to-day design.

2.2.1 Loss of Ground

Excavation of a tunnel in soft ground generally causes the soil to displace inwards across the tunnel perimeter. This volume of soil (V_t) is termed 'lost ground' and is expressed as a volume per unit length of tunnel. The percentage of lost ground, V_t (%) is the volume of lost ground expressed as a percentage of the total tunnel volume. These definitions were offered by Cording and Hansmire (1975:575).

Traditional viewpoints of tunnelers have always linked settlements to the loss of ground. Procedures for calculating the loss of ground have been studied by Hansmire (1975), but most of them require extensive field measurement results. A simplified empirical procedure which has been presented and checked against field measurements by Cording and Hansmire (op.cit.) will be used in this thesis. These authors verified that the volume loss into tunnels could be estimated from deep vertical displacement measurements by an expression of the form:

$$V_t = 2\delta(a+y) \tag{2.1}$$

where V_t is the volume lost into the tunnel (m^3/m), δ is the deep vertical displacement measured at a distance y above the crown. According to Cording and Hansmire (op.cit.), the distance y has to be small with respect to the tunnel radius a, otherwise the equation will misestimate the loss of ground.

2.2.2 Overload Factor

In a classical paper, Broms and Bennermark (1967) postulated that the stability of a vertical clay face of a hole in a retaining wall could be defined by an expression of the form:

$$OF = (P-P_i)/c_u \tag{2.2}$$

The parameter OF is frequently termed 'overload factor' (Clough and Schmidt, 1981). For the case of a tunnel, P would be the overburden pressure at the tunnel centerline (γz) and P_i would be the value of an internal pressure (e.g.

compressed air or mud pressure). The c_u parameter stands for the undrained shear strength in the zone adjacent to the exposed tunnel face.

The tests carried out by Broms and Bennermark (op.cit.) were aimed at investigating the intrusion of clay at depth into vertical openings. They verified that instability occured at a load 6 to 8 times the undrained strength of the soil, i.e., for OF=6 to 8. The value OF=6 is frequently taken as the possible onset of tunnel face instability. Schmidt (1969:61) suggests that this limit could be lower than 6 for the case of shallow tunnels. Ward and Pender (1981:23) state that for tunnels with small cover to diameter ratios, the value of 6 is on the unsafe side.

Strictly speaking, the c_u value (and consequently the OF value) should only be considered for classes of problems involving saturated clays and where little volume change is expected during the tunnel excavation process. Although it is the simplest strength parameter obtainable, it is very dependent on the test procedure, as described by Lambe and Whitman (1969:Chapter 28). Moreover, an undrained condition is only practically achievable in laboratory tests in which impervious boundary conditions are physically imposed to the soil sample. The c_u value should thus be regarded more as an indicator than a proper parameter.

Appropriate judgement should then be exercised when involving the OF values in design considerations. Nevertheless, its use appears to be widespread among practitioners and it seems difficult to dismiss it in tunnel engineering at the present time.

2.2.3 The Settlement Trough

The soft ground tunnel excavation and consequent ground losses normally provoke the development of a depression of the surface of the ground located above the tunnel which is frequently termed 'settlement trough'. It has been shown that this trough resembles the normal probability curve or error function (Schmidt, 1969; Peck, 1969a).

Schmidt's approach considered the soil medium in terms of a stochastic model and did not take its stress–strain behaviour into account. This fact has been criticized (e.g. de Mello, 1981) but the use of Schmidt's theory, mainly as a method for predicting surface settlements, is still frequent. This is likely due to its simplicity and to the fact that most subsidence profiles indeed bear likeness to the error function.

A trough resembling the error function is depicted in Figure 2.1, which summarizes its properties. The maximum settlement at surface is termed S, and the settlement at any location is defined by S. The inflection point (i) is termed 'trough width parameter'.

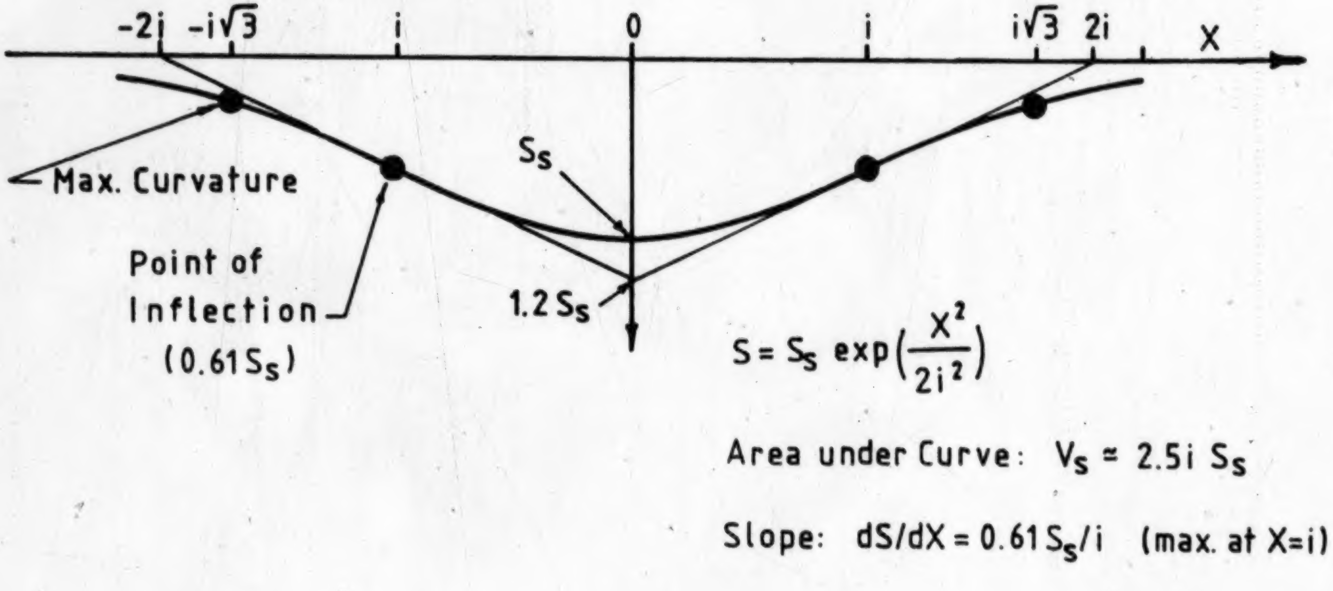

-2i
-i√3
i
0
i
i√3
2i
X
Max. Curvature
Point of Inflection (0.61 Ss)
Ss
1.2 Ss
S = Ss exp(X²/2i²)
Area under Curve: Vs ≈ 2.5i Ss
Slope: dS/dX = 0.61 Ss/i (max. at X=i)

Figure 2.1 Settlement trough represented by the error function

2.3 NATM Performance

In this section, data collected from case histories are used to position the NATM works with respect to tunnels constructed by other methods. In order to permit a direct understanding of the concepts reviewed, use is made of diagrams which utilize some of the indices reviewed above.

2.3.1 Overload Factor vs. Ground Loss

Based on theoretical analysis for closure into a deep opening in homogeneous clay, Schmidt (1969) has obtained relationships between the overload factor and ground loss. Schmidt has placed his curves on a diagram where points representing specific case histories were added.

The theoretical curves may be interpreted as an upper limit above which plastic deformations will take place and excessive ground losses may occur. The two curves bound a narrow range of soil undrained strength/undrained deformation modulus (c_u/E_u), as shown by Clough and Schmidt (1981:618).

Clough and Schmidt (op.cit.) have reviewed Schmidt's diagram and proposed newer theoretical curves for the 'unrestricted' range of ground loss, adding other case histories as well. The reviewed diagram is reproduced in Figure 2.2. Also shown in this figure are the results of centrifugal model studies in normally consolidated kaolin presented by Mair et al. (1981). These tests were carried out with tunnels geometrically similar but at different depths and show that the cover/diameter ratio (C/D) is a key factor in the evaluation of ground loss. While the 'deep' tunnel (C/D=3.11) values fall within the theoretical range, the 'shallow' tunnel (C/D=1.67) shows a larger ground loss for the same OF value. This suggests that the indicator OF is not proper because it does not account for the C/D ratio.

It is interesting to evaluate the potential ground losses with respect to OF ranges. The following ranges can be recognized[5]:

5. **OF<2:** The theoretical potential ground loss is less than about 1%. These ground losses can be considered small and usually produce inconsequential settlements.
6. **2<OF<4:** Potential ground loss might reach magnitudes of 10%. Ground movements through the face are usually small. Adjacent structures might be affected by settlements.
7. **4<OF<6:** Potential ground loss is above 10%. Radial and face displacements become significant. For OF values near 6, the face displacements are of such magnitude that face support (e.g. compressed air) might be required.

Clough and Schmidt (op.cit.) have further observed that the actual ground loss may be greater than the theoretical value given by Schmidt's theory because of loosening along fissures in overconsolidated clays. This will also be the

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[5] This classification has been adapted to the present study from the work of Clough and Schmidt (1981).

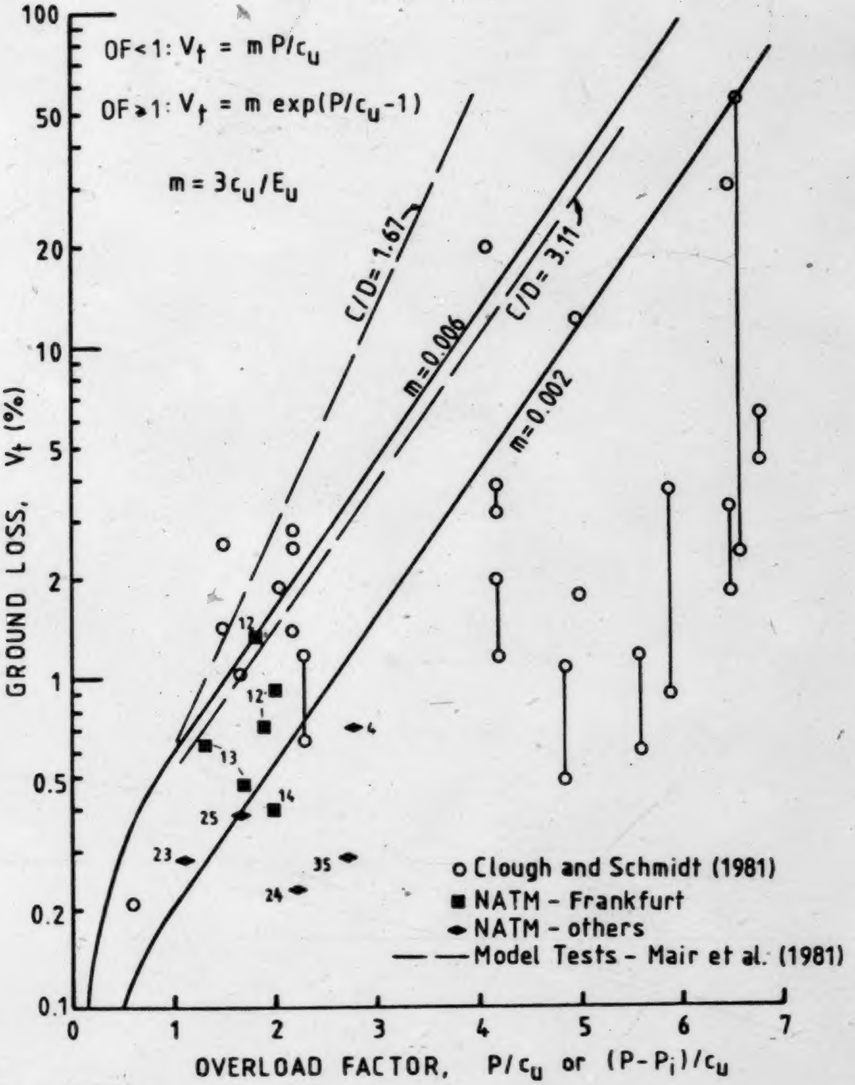

Figure 2.2 Relationship between overload factor and loss of ground (numbers refer to cases listed in Appendix B)

case of tunnels where some volume change takes place. Also, the proximity of the surface boundary may have the effect of increasing the loss of ground for a given OF, as shown by the experimental studies published by Mair et al. (op.cit.).

NATM case histories for which sufficient data were available have been added to the inventory published by Clough and Schmidt (op.cit.). The values for the overload factor OF and for the loss of ground $V_t(\%)$, as well as the list of cases corresponding to the numbers in the figure can be found in Appendix B.

No specific trend can be recognized for the NATM cases reviewed and the ground losses appear to be comparable to those induced by the shield tunnels reported by other authors. The Frankfurt cases (C/D varying from 1.04 to 1.94) fall very close to the theoretical bounds, possibly due to the proximity of the surface boundary.

An important observation is the fact that the majority of the NATM cases fall in the region where OF≤3. On the other hand, it should be pointed out that some variability of the OF values has to be expected. Points representing the Frankfurt cases for example were calculated with basis on an average c_u of 150kPa. A study by Katzenbach (1981:68) reports lower limits of 50kPa for the Frankfurt clay, which would yield OF values of about 5 for the cases studied. Nevertheless, this is an indication that the method might be inadequate in cases where potential ground losses through the face are significant, which is in agreement with Atrott and Sauer (1983:17). These authors state that the application of the NATM is dependent upon the 'ground stability', necessity for ground improvement measures existing otherwise. At least one NATM case was examined where OF before any ground improvement was above 6 (Cruz et al., 1982; Celestino et al., 1982). Some very large deformations were observed in an area where the ground control measures were inefficient.

2.3.2 Trough Width vs. Depth/Width Ratio

By combining results from both theoretical stochastic and elastic analysis, Schmidt (1969) derived a relationship for the trough width parameter i:

$$i/a = k(z/2a)^n \qquad (2.3)$$

where a is the tunnel radius, z is the depth to the tunnel axis, the coefficient k near unity and the power n approximately 0.8. In the derivation of this equation, Schmidt assumed that the volume changes in the subsiding mass could be neglected.

Schmidt (op.cit.) and Clough and Schmidt (1981) have presented empirical evidence which shows that equation 2.3 holds true for clays mainly. Attewell (1978:883) has sought other relationships and found k=1 and n=1. This would yield i=z/2, which can be used as a 'rule-of-thumb'. The validity of equation 2.3 was also analysed by Mair el al. (1981), based on centrifugal model tests. Their results suggest that equation 2.3 (with k=1 and n=0.8) would overestimate the trough width parameter for the case of stiff clays and underestimate this parameter for the case of soft clays.

Figure 2.3 (adapted from Clough and Schmidt, op.cit.) shows trough widths (i/a) as a function of the depth-width ratio (z/2a) for a number of tunnels in clay. Also shown are the simplified versions of equation 2.3 proposed by Attewell and by Clough and Schmidt. On the same figure the measured distortions in some of the NATM cases have been superimposed. Because settlements and trough sizes are controlled often by complex ground conditions, predictions can never be precise. Nevertheless, it is apparent that the quantity i for 'regular cross section' tunnels [*] tunnels in clay can be estimated by any of the simplified forms of equation 2.3. This indicates that the method produces trough widths comparable to those caused by other tunnelling procedures.

Cases 28a, 28b and 28c correspond to tunnels with large cross sections of the Munich underground system. As will be shown in Chapter 3, a feature sought in some of these works is a flat surface settlement trough. This achievement is confirmed by the location of these points in Figure 2.3. It is apparent that the trough width vs. depth correlations

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[*] 'Regular cross-section' tunnels are defined herein as those which do not exceed the dimensions of conventional single-track subway tunnels (normally 30 to 40m^2 of cross-sectional area). 'Large cross-section' tunnels are those whose cross-sectional excavated area exceeds 60m^2, reviewed in Chapter 3.

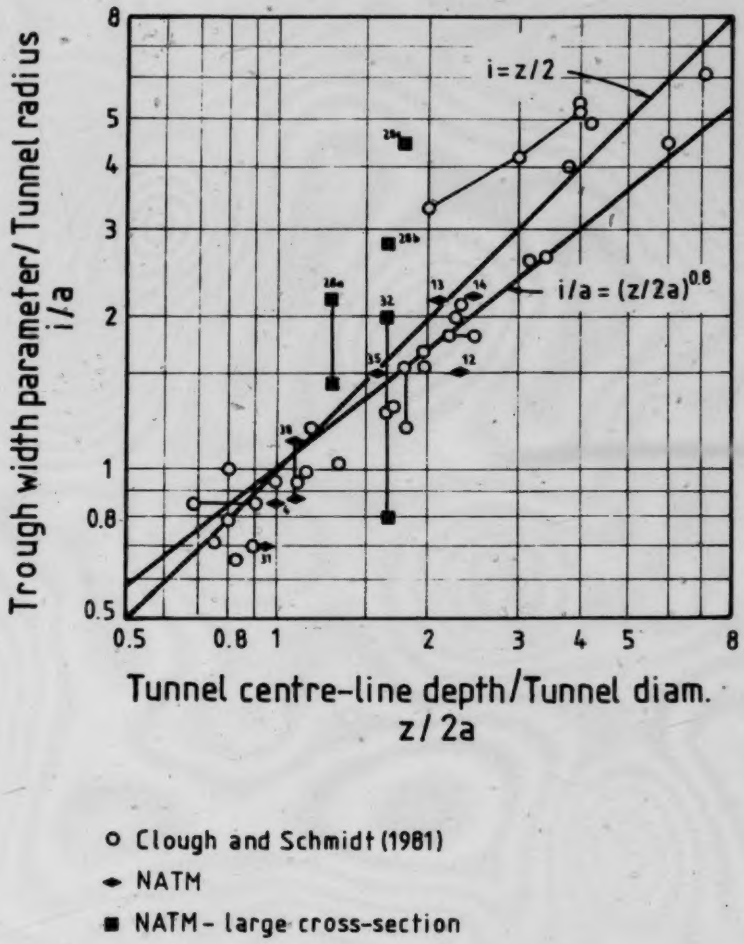

Figure 2.3 Settlement trough width parameter i vs. tunnel depth z in normalized logarithmic diagram (after Clough and Schmidt, 1981:modified; numbers refer to cases listed in Appendix B)

traditionally applied to regular cross-section tunnels may not be applicable to the large cross-section ones.

Case 32 is also a large cross section tunnel, driven in tertiary marls. The trough was highly assymetric and exceptionally steep on on one side. Egger (1975) attributes this feature to geological particularities of the site.

2.3.3 Quality of Construction

The influence of the quality of construction on the displacements caused by tunnel excavation has been recognized for some time (e.g. Peck et al., 1972). Based on suggestions by Peck et al. (op.cit.), Negro (1979) has tried to incorporate the parameter 'quality of construction' into the procedures for assessment of tunnelling performance.

Negro's work is used herein as a way of classifying construction quality of NATM when compared to other methods. Figure 2.4, modified from Negro (op.cit.) shows curves representing four levels of construction quality. In this Figure, V_s is the volume of the settlement trough (m^3/m) at the surface, which can be determined from field measurements or approximated by the error function properties outlined in Section 2.2. V_{tn} is the volume per meter of the excavated tunnel. Some Edmonton case histories (shield tunnels) have been added to Figure 2.4 for illustration purposes.

Negro (op.cit.) proposed four levels of construction quality, expressed by the curves shown on Figure 2.4:

1. V_s=0.50%V_{tn}: High construction quality

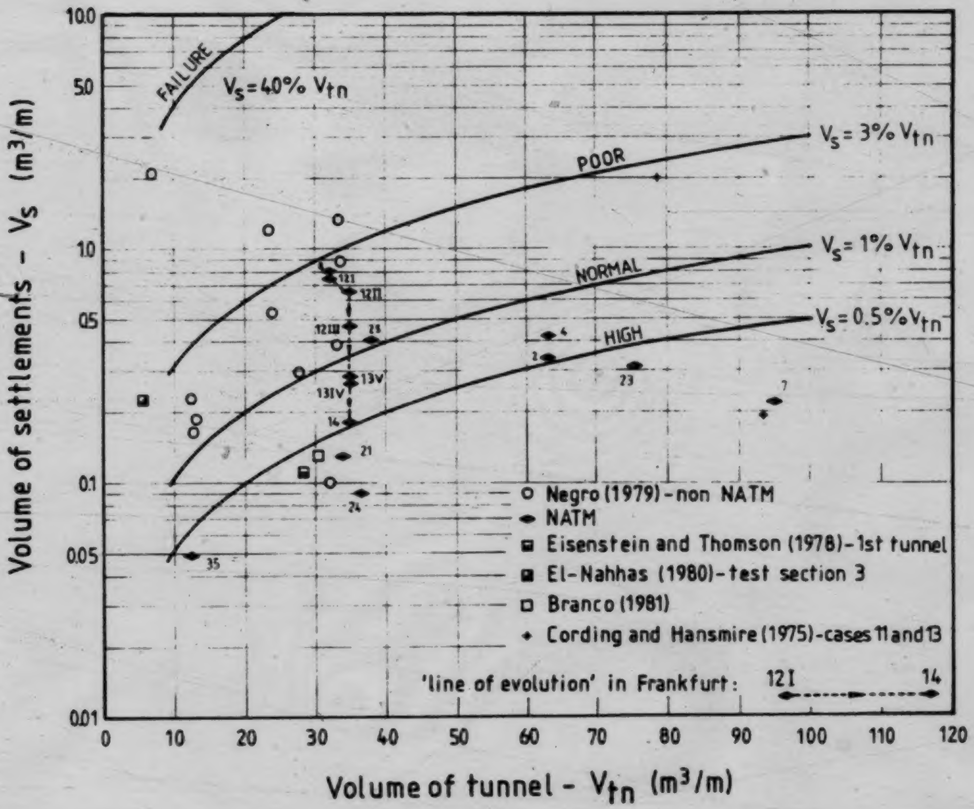

Figure 2.4 Construction quality assessment (numbers refer to cases listed in Appendix B)

2. $V_s=1.00\%V_{tn}$: Normal construction quality
3. $V_s=3.00\%V_{tn}$: Poor construction quality
4. $V_s=40.0\%V_{tn}$: Failure

Although these levels might appear rather arbitrary, they can be related to the maximum surface distortions by applying a procedure outlined by Clough and Schmidt (1981:628). Figure 2.4 enables a qualitative appreciation of the construction quality involved in each work. Some of the NATM cases have been placed on the diagram. Also shown are the tunnels surveyed by Negro (op.cit.).

The number of case histories examined is limited and any conclusion that can be derived must be considered tentative. Nevertheless, it is noticeable that all the NATM cases examined fall below the limit specified for poor construction quality. Possible bias exist due to the fact that cases where excessive displacements occurred are seldom published in detail, but this might be the case not only for the NATM projects but also for the shield driven tunnels.

Another interesting feature is the 'line of evolution' for the tunnels in Frankfurt indicated in Figure 2.4. Point 12I represents the first instrumented section of the first NATM project in that city (Baulos 25 - works started in September/1969 according to Edeling and Schulz, 1972:358), points 12II and 12III representing subsequent instrumented sections of the same tunnels. Points 13IV and 13V represent a posterior contract section (Baulos 17), initiated about a year later (Blindow et al., 1979:217). Point 14 corresponds to the Baulos 18a lower tunnel (see description in Appendix A), which was initiated in 1973 (Blindow et al., 1979:218). All tunnels were excavated in a similar soft-ground medium.

The analysis above shows that the quality of construction in the Frankfurt tunnels improved considerably in a period of 3-4 years. This is likely due to acquaintance gained with the method and with the ground, associated with the flexibility to change construction features in the NATM. It is thus apparent that the NATM is a procedure which allows for an improvement in construction quality, reflecting perhaps the requirement of some startup time until an optimum geotechnical performance can be achieved.

2.4 Ground Displacements around NATM Tunnels

In urban areas, one of the major issues associated with tunnelling construction is the influence that the process of excavation and lining placement will have in the shape and magnitude of the surface and subsurface settlements. Excessive differential settlements may cause damage to existing buildings and utilities. It is consequently of paramount importance that designers and contractors be aware of factors that may induce settlements so that adequate standards can be established.

The information collected from NATM case histories provided means to recognize some of the main factors affecting the development of the ground movements with face advance. It was seen that the displacement development was closely linked to some construction details which will be described in the following section. It was thus possible to identify the main sources of loss of ground around tunnels excavated by the NATM.

2.4.1 Construction Factors affecting Displacements

As reported in Section 2.2, ground displacements are normally associated with the traditional concept of 'loss of ground'. Attempts have been made to identify the sources of loss of ground around tunnels (e.g. Hansmire, 1975; Cording and Hansmire, 1975). Emphasis however was placed on shield projects.

In an earlier work, Schmidt (1969) was able to evaluate sources of ground movements around tunnels in a more general manner. His evaluation was based on theoretical considerations and on analysis of case histories which included shield and non-shield tunnels and seems very appropriate to the present study. Schmidt (1969:73) concludes that the loss of ground is a function of:

1. the distance from the face at which the ground is supported;
2. the quality of the contact between the lining and the soil;
3. the capability of the lining to resist crown settlements and wall movements;
4. any efforts to restrict face movements.

Schmidt (op.cit.) further points that minimization of the loss of ground is generally achieved in construction procedures that limit the ground movements at an early stage.

The quality of the contact at the soil-shotcrete interface is normally considered very good (Simondi et al., 1982). When working within the framework proposed above one is then inclined to associate displacement development in NATM tunnels with factors 1, 3 and 4 listed above. Their relative influences on the total amount of settlements will be examined in the following items.

Papers that deal specifically with the factors affecting the displacements around NATM excavated tunnels have been published in Europe by many authors (e.g. Müller, 1978b, 1979 or Laabmayr and Pacher, 1978). These works show the importance of limiting movements around shallow tunnels at an early stage, stressing the importance of factors like the 'invert closure distance' and the 'invert closure interval'. This terminology is defined in the following paragraphs.

2.4.1.1 Invert Closure Distance – Invert Closure Interval

In shallow soft ground tunnels it is important to close the invert quickly to form a load bearing ring and to leave no section of the tunnel unsupported (Brown, 1981:16). The invert closure distance (L) is defined as the length between the face of the heading and the point at which the shotcrete ring is fully completed. The corresponding time interval between completion of one excavation round and this invert closure has been termed 'ring closure interval' or 'invert closure interval' (T).

The importance of these two factors, although already stressed in the literature, is analysed more closely in the present work, by using data collected from the review of case histories. For most of the tunnels examined, records of short term surface settlements were the only available information regarding the lost ground. Although it is recognized that volume changes between the crown and the surface occur frequently, it may be assumed that there is a direct relationship between the ground lost into the tunnel and the surface movements.

In Figures 2.5 and 2.6, correlations between the values of L and T and the surface settlements are attempted. The settlements have been normalized as per procedure described by Oteo and Sagaseta (1982), in order to minimize discrepancies due to different geometries and soil properties. Since the values of these normalized settlements depend highly on the value of the E modulus, a range is presented for each case. Limitations due to representing the soil behaviour by

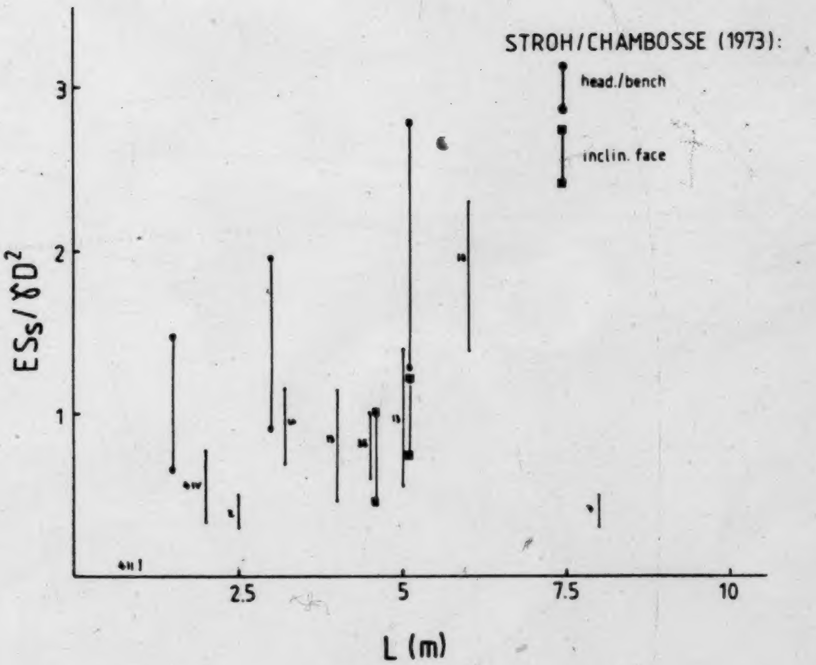

Figure 2.5 Invert closure distance vs. surface settlements

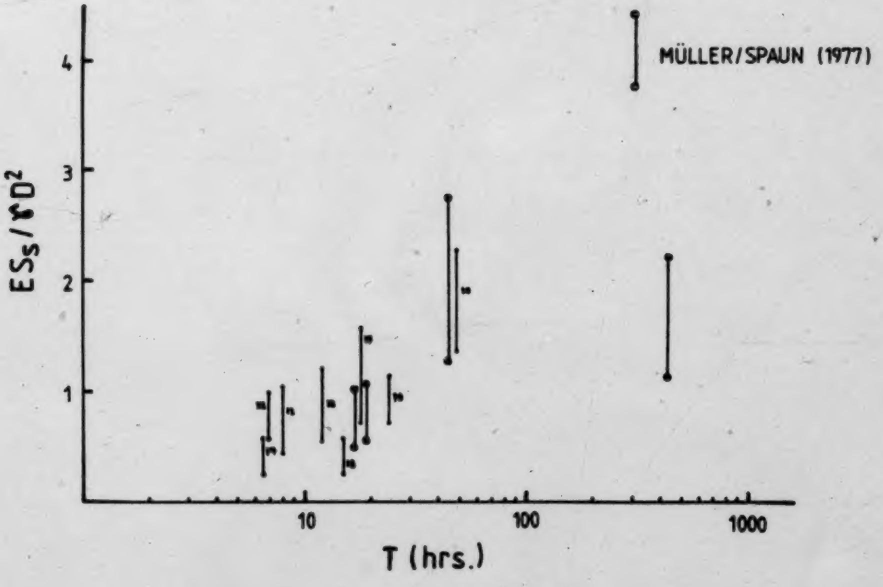

Figure 2.6 Invert closure interval vs. surface settlements (numbers refer to cases listed in Appendix B)

the E value solely are obvious but in most cases it was the only parameter available. The procedures to obtain the E value may vary from case to case as well.

Figures 2.5 and 2.6 show a tendency of the maximum surface settlement to increase as L and T are increased. Despite the shortcomings reported above, it is believed that the trends observed in these figures mirror the actual behaviour in the field. It is obvious that these two factors are related because a rapid invert shotcreting will likely be achieved if the excavation rounds (and consequently the invert closure distance) are short.

An implicit factor which is related to the value of T is the degree of time dependent behaviour of the ground. Causes of time dependent behaviour around tunnels are frequently connected to the presence of ground water. The trend apparent from Figure 2.6 (which includes tunnels excavated above and below the water table[7]) indicates that in settlement analysis of NATM excavated tunnels an assessment of the degree of time dependency of the ground is desirable. That is, in time dependent ground the rate of construction and consequently the invert closure interval would play an important role in the amount of displacements caused by tunnel excavation.

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[7] With ground water lowering prior to construction.

2.4.1.2 Depth of Heading Excavation

Stroh and Chambosse (1973) have presented data from eight measurement sections in Frankfurt subway tunnels. Their data is most valuable because all tunnels were built under similar conditions, in the same soft-ground environment. Figures and tables presented by these authors allow us to identify the following components of the total crown settlement (St), measured by extensometers:

1. Sf: crown settlement ahead of the face, i.e the crown vertical displacement when the face of the heading reaches the test section.
2. Sm: crown settlement occurring after the face reaches the test section but before the invert is closed.
3. Sn: crown settlement occurring after the invert is closed.

In addition to these values, Stroh and Chambosse (op.cit) present data regarding the depth of heading excavation (La) and the average rate of excavation (v). These values are summarized in Figure 2.7. Also shown in this figure are tentative correlations between La and Sf as well as La and (St-Sf). It is apparent that the longer the heading excavation La is, the larger the settlements Sf tend to be. The value of La does not appear to correlate with the remaining crown settlement St-Sf. Although there is a possible bias due to the time

MQ	v (m/d)	La (m)	L (m)	Sf (mm)	Sm (mm)	Sn (mm)	St (mm)	REMARKS
I	1.7	5	9.5	38	13	10	61	head.-bench
II	2.2	3	6.0	14	15	13	42	head.-bench
III	1.7	1.5	2.5	13	9	12	34	head.-bench
IV	2.0	1.0	5.0	6	14	9	29	incl. face
V	3.0	1.0	4.0	4	9	9	22	incl. face
I-2	1.8	4.0	7.5	-	-	9	59	head.-bench
I-4	1.8	5.0	9.5	-	-	8	70	head.-bench
I-7	1.8	4.0	7.0	-	-	11	52	head.-bench

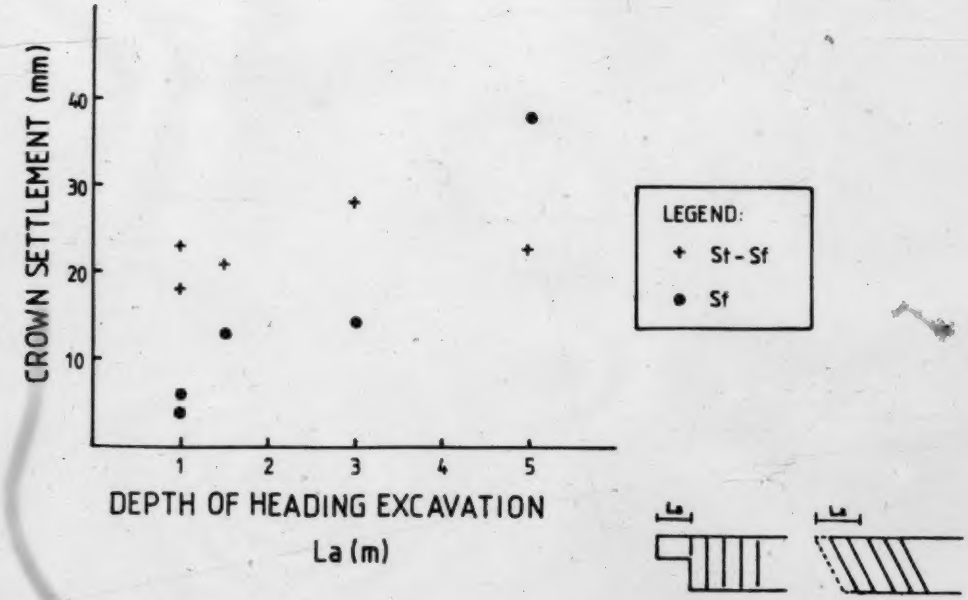


Figure 2.7 Settlement components for Frankfurt tunnels

dependent properties of the ground, this could indicate that the loss of ground through the face can be minimized by excavating small heading depths.

2.4.1.3 Lining Deformability

Information regarding the overall stiffness of the shotcrete lining is very seldom reported in the literature. Analysis of several case histories shows that the schotcrete thickness in linings of regular cross-section subway tunnels (i.e 30 to 40 m^2 of cross-sectional area) normally falls in the range 15-25cm. The rib spacing however is frequently shortened when underpassing buildings or other major structures (e.g. Jagsch et al., 1974).

Numerical studies by Lombardi and Amberg (1979) indicate that the displacements after the invert is closed can be minimized by increasing the lining stiffness. This increase in stiffness should not be achieved by increasing the lining thickness but rather by decreasing the rib spacing, to avoid cracking effects due to generation of bending moments in a thick walled lining (see Müller, 1978b:29).

Another source of concern appears to be the stiffness of the lining at the tunnel heading. Use has been made of mini-beams placed longitudinally between ribs (e.g. case of Essen Los 24a in Appendix A). It may be speculated that these beams will increase the longitudinal stiffness of the lining and thus minimize the crown displacements occurring after bench removal but before the invert is closed. However, no specific study about this problem has been found in the literature.

Information about deformations due to changes in the lining diameter associated with thermally induced shrinkage or creep strains was not available in the case histories examined. Indication that the shrinkage problem is present exists (e.g. Heilbrunner, 1980:216), but the concern appears to be more related to fissuring of the shotcrete than to ground losses.

2.4.2 Quick Estimates of Ground Losses

The engineer is frequently faced with situations where decisions which do not allow any delay have to be made. The present section is included for the benefit of those who have a job to do but who do not have time for elaborated calculations.

It is recognized that the following conclusions are based on a statistically limited and probably biased population (cases with large ground losses are seldom reported). Nevertheless, the following figures may be constantly improved by adding the results of new experiences. It is believed that they may represent estimates of NATM ground losses under normal conditions (i.e. little groundwater infiltration; good, experienced workmanship).

Examining the field data presented in Appendix B, one can idealize a prototype NATM single tunnel with regular cross section and with the following characteristics:

a. diameter D;
b. invert closure distance $L \leq 0.5D$;
c. shortest possible invert closure interval;
d. heading excavation length $La \leq 0.5D$ with shotcrete lining protecting exposed roof;
e. shotcrete thickness 15–25cm, rib spacing 0.8–1.2m.

If such a tunnel is driven under a condition where little face movements take place (say $OF \leq 3$, see section 2.3.1), it is believed that the following ground losses can be safely estimated:

a. face loss: $Vf(\%) \leq 0.3$
b. unsupported heading: $Vm(\%) \leq 0.3$
c. lining deformation: $Vn(\%) \leq 0.3$

The sum of these ground losses will yield a total which is less than 1% of the tunnel volume, a value commonly adopted for shield projects (Attewell, 1978:881/882).

2.4.3 Distribution of Ground Movements

There have been several attempts to establish a relationship between the amount of ground that is lost close around the tunnel (V_t) and the amount of ground that is lost at the surface (V_s) during tunnel excavation. This relationship is of interest to the practicioner because of its importance in procedures for estimating settlements. Also, comparisons of the differences between the volumes V_t and V_s have been proven illustrative of the strain field which develops in the soil mass around the tunnel (Hansmire, 1975; Cording and Hansmire, 1975).

2.4.3.1 Volume Changes

Hansmire (op.cit.) has presented data from shield tunnels in a very simple diagram reproduced in Figure 2.8, which shows NATM cases where sufficient data was available. The solid diagonal line represents the $\Delta V=0$ condition, the volume lost about the tunnel being equal to the volume of surface settlement. This line separates areas representing conditions where tunnelling is accompanied by increases in soil volume ($\Delta V>0$) or by decreases in soil volume ($\Delta V<0$).

Traditional views have associated these three conditions with the natural state of the soil mass. Hansmire (1975:192) states that the $\Delta V>0$ domain is characteristic of medium to dense granular soils, while $\Delta V<0$ may be expected for the case of very loose granular soils. He further points that $\Delta V=0$ may be expected for a tunnel in soft saturated clay.

It can be noted that most of the NATM points fall below the diagonal line. This could be due to an inadequacy of the empirical correlation by Cording and Hansmire (Equation 2.1) or to imprecise measurements, but may also indicate an average 'compressive' behaviour above these tunnels. Cases published by Cording and Hansmire (1975) have been added to the diagram. These shield driven single tunnels generally plot in the $\Delta V>0$ domain.

Cases 12 (I, II and III) and 13V correspond to twin tunnels driven in a tertiary clayey marl. It is apparent that for these twin NATM tunnels the behaviour postulated by Cording and Hansmire (op.cit.) for twin shield tunnels holds true. According to these authors, disturbance of the soil around the first tunnel would increase soil compressibility, contributing to volume decreases. For sands, in addition to this effect, volume decreases would develop in the previously expanded zone over the first tunnel (Cording and Hansmire, 1975:629). This could also be the case for the 'simultaneously' excavated tunnels, possibly because there is always a certain distance allowed between the excavation faces, one of the tunnels being therefore the 'second' to be excavated.

The tendency of NATM cases to fall in the $\Delta V<0$ domain might indicate that tunnels excavated by the NATM create displacement fields which are different from those caused by shield driven tunnels. Since the assumption $V_s=V_t$ is often taken as 'safe' for settlement prediction purposes, care should be exerciced in NATM projects, which might actually show a slight tendency to

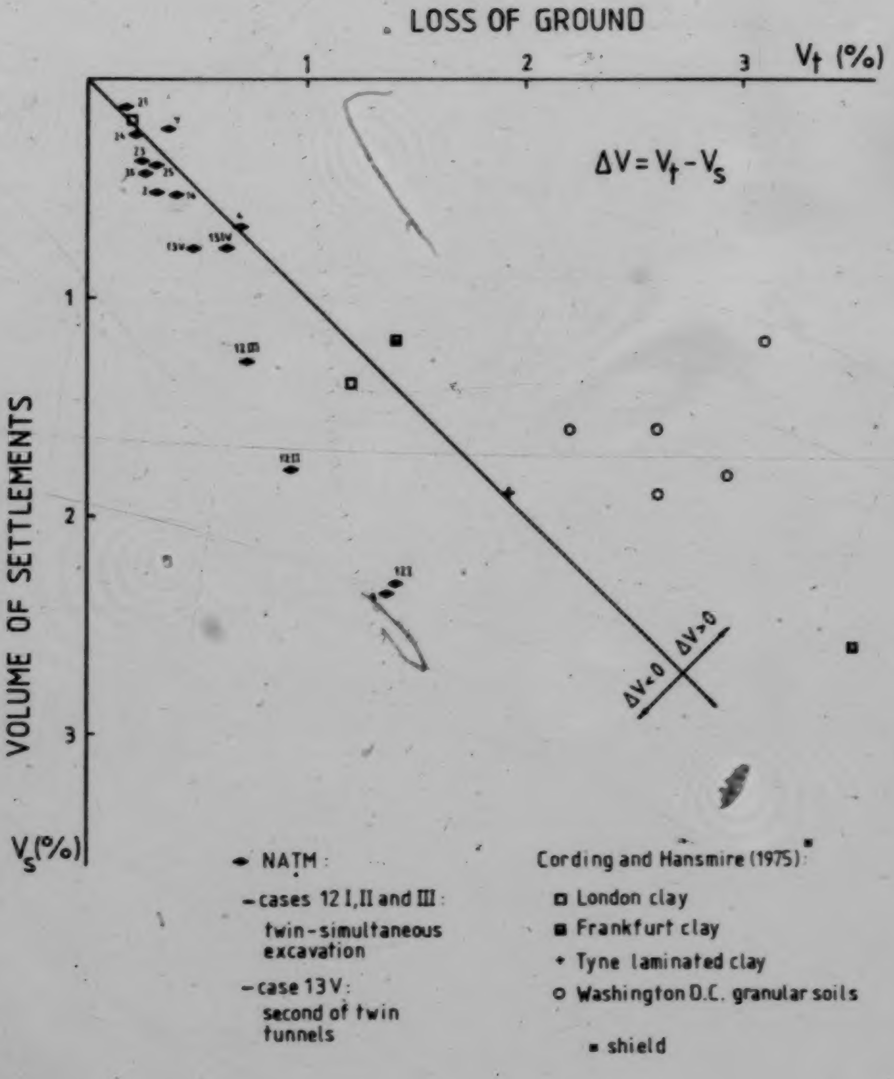


Figure 2.8 Soil volume changes in Hansmire's diagram (numbers refer to cases listed in Appendix B)

fall in the $\Delta V<0$ domain. The assumption $\Delta V=0$ would then lead to underestimation of surface settlements in methods which make use of this concepts.

2.4.3.2 Surface vs. Deep Settlements

The relationship between V_t and V_s may also be determined by simple equations derived by Atkinson and Potts (1977), which were based on results from model tests. Since the ground displacements above the tunnel crown are normally largely vertical, these authors sought a relation between the vertical displacement above the crown S_t and the maximum surface settlement S_s, which normally occurs over the tunnel axis. The relationship derived suggests a linear correlation between the ratio S_s/S_t and the cover/diameter (C/D) ratio:

$$S_s/S_t=1-\alpha(C/D) \qquad (2.4)$$

In this expression, α is a measure of the dilation of the ground. Some values of α presented by Atkinson and Potts (op.cit.) are:

a. $\alpha=0.57$: dense sand at low stresses
b. $\alpha=0.40$: loose sand and dense sand at large stresses
c. $\alpha=0.13$: overconsolidated kaolin

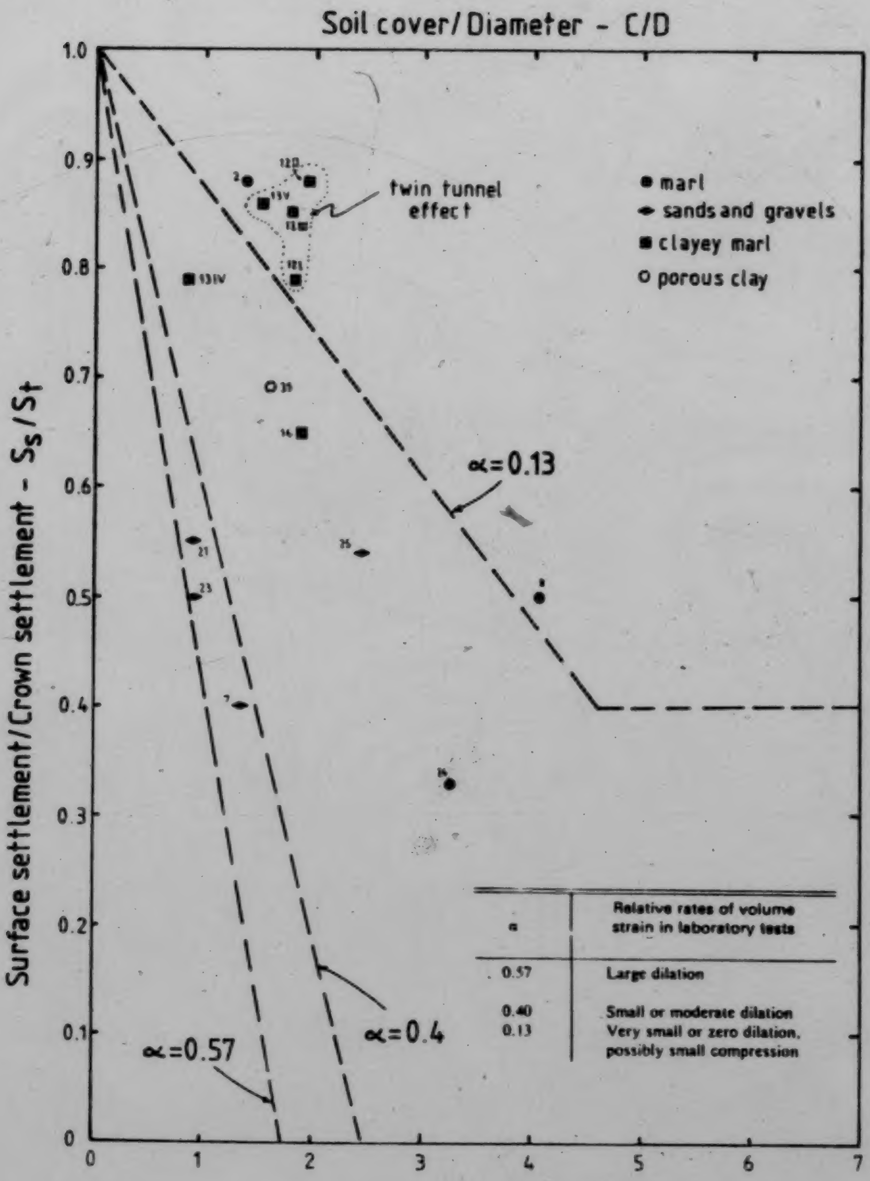
Assuming that the invert remains stationary, that the magnitude of the crown settlement is small compared to the tunnel diameter and that the surface settlement trough obeys Schmidt's theory (Section 2.2.3), Atkinson and Potts (op.cit.) derived an equation of the form:

$$V_s/V_t \simeq 1.60(i/D)(S_s/S_t) \tag{2.5}$$

Equations 2.4 and 2.5 could be employed to estimate relationships between the volumes lost at the tunnel and at surface. However, field data presented by Ward and Pender (1981) shows that the relationship expressed by equation 2.4 is only approximate at best.

Figure 2.9 shows data from the NATM case histories plotted on a diagram similar to those presented by Atkinson and Potts (op.cit.). It appears that the correlations presented by these authors are not applicable to the case histories examined. However, two features can be observed:

1. The α=0.57 and α=0.40 lines bound a lower limit and some of the sands and gravel sites lie close to this limit. An approximate lower bound considering the cases examined would be given by α=0.5.
2. The cases which have the 'twin tunnel effect' described in the preceeding section lie above the α=0.13 line. This suggests that in this cases there is only a very small or zero 'average' dilation, most likely some contraction.



α	Relative rates of volume strain in laboratory tests
0.57	Large dilation
0.40	Small or moderate dilation
0.13	Very small or zero dilation, possibly small compression

Figure 2.9 NATM field data vs. relationships derived from model tests by Atkinson and Potts, 1977 (numbers refer to cases listed in Appendix B)

2.4.3.3 Surface Settlement ahead of the Face

An indication of the geotechnical performance of soft-ground tunnels is the slope of the surface settlement profile perpendicular to and ahead of the tunnel. These measurements are seldomly reported in the literature, possibly because the transverse settlement trough is generally of greater concern.

Some indication exists that the distance of influence ahead of the face (herein termed Ld) in NATM excavated tunnels is larger than that found in shield and other projects. In Frankfurt, Chambosse (1972:18) reports Ld values of about 2.2 tunnel diameters for the NATM and Ld≃1.3 diameters for the shield case, driven in the same environment. Negro and Eisenstein (1981:Figure 12) report values of Ld≃2.6 diameters for the NATM and Ld≃1 to 1.6 diameters for two other non NATM tunnels excavated under similar conditions.

2.5 Assessment of NATM Limitations

The adopted method of excavation and construction of a tunnel must be appropriate to the ground conditions so that it is possible to advance the tunnel safely, maintain its integrity at least temporarily and avoid damage to nearby structures (Peck, 1969a:226). An assessment of the feasibility of tunnelling by means of simple parameters like the OF value presented in section 2.2.2 is atractive, but may not be appropriate in every case. In stiff fissured clays for example, a tunnel could have a low OF value derived from laboratory test results in small samples. The overall ground behaviour however could be controlled by the presence of fissures or localized geological features like saturated sand pockets. This tunnel could present stability problems such as those described by Matheson (1970).

Reports of accidents would be most interesting to assess the degree of confidence that can be placed in a method. Regretably, unfortunate experiences are seldom published with the clearness necessary for elaborating an appropriate analysis. A few cases reported associate the failures with unpredicted or wrongly interpreted geological and geotechnical conditions, frequently connected with some construction deficiency (e.g. Blindow and Wagner, 1978; Müller, 1978b; Hereth, 1979; Golser and Hackl, 1981). Inadequate ground control could also play a role in causing damages (e.g. Celestino et al., 1982; Cruz et al., 1982), sometimes in combination with non-appropriate site investigations (Nixdorf, 1980:145).

A first impression from the description of failures reported in the references quoted above is that eventual damages could be avoided by following certain basic principles. The following list has been adapted from Golser and Hackl (1981:213) seeming appropriate for the case of NATM shallow urban tunnels:

1. Execution of a thorough site investigation during the planning phase.

2. Inclusion of geological and geotechnical ground characteristics as a vital element in design.
3. Establishment of a flexible contract which can allow modifications during construction without litigation problems.
4. Establishment of a monitoring program which will permit the application of an 'observational method of design' following the concepts proposed by Peck (1969b).

The Author is also of the opinion that the chances of an accident could be minimized by carefully preparing a complete geological longitudinal profile along the proposed tunnel route following the site investigation phase. Critical points to be encountered during excavation of the tunnel would be identified in such profile as described by Krischke and Weber (1981:125).

2.6 Summary and Conclusions

In this chapter, data collected from case histories was used to classify the NATM tunnels according to indices commonly found in engineering practice. Construction factors affecting the ground movements were also outlined.

It is recognized that published studies, although providing information on the character of the ground movements, offer only limited insight into the actual mechanisms generating these deformations. Also, the population examined was statistically limited and probably biased due to the fact that 'non-successful' ventures are seldom reported in sufficient detail. Notwithstanding all these considerations, some important observations can be made. It is believed that their validity may be reinforced in the future by means of a large number of field observations in a variety of ground conditions and perhaps by means of numerical studies.

The most important observations derived from the analyses presented in this chapter are:

1. The NATM might not be an appropriate method of construction for situations where expected face displacements are significant unless effective ground improvement measures are used.
2. The NATM regular cross-section soft ground cases examined presented a geotechnical performance comparable to shield tunnels. However, indication exists (sections 2.4.3.1 and 2.4.3.3) that the NATM excavated tunnels create displacement fields which are different from those caused by shield tunnels. The study of displacement fields around NATM tunnels is undoubtedly an area of research that could be pursued in future works. This would require however an extensive instrumentation of one or more case histories.
3. The NATM is a method which allows for an improvement in the geotechnical performance due to flexibility in changing construction features[*]. Some startup time may be required in order to achieve an optimum geotechnical

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[*] This will, of course, require a correspondent flexibility in the contractual aspects.

performance in a new environment, as shown by the examples of the tunnels in Frankfurt.

4. Ground losses can be minimized by reducing the invert closure distance, the invert closure interval and the depth of heading excavation.
5. Large cross-section tunnels (described in the next chapter), which are normally excavated in stages, appear to cause displacement fields which apparently differ from those related to regular cross-section tunnels. More field instrumentation results and perhaps numerical analyses are needed for better understanding of the ground behaviour around such structures.

3. LARGE CROSS-SECTIONS

3.1 Introduction

Modern underground systems in urban areas involve cross-section forms which are not always adaptable to shield tunnelling. Typical examples are underground stations, junctions of lines and double track tunnels. It is probably in these applications that the NATM proves most useful. This is due to the possibility of excavating a variety of cross-sectional shapes and to the flexibility of changing excavation layouts according to necessity.

In this chapter attention is focussed on construction of double track subway tunnels and subway stations. Within the scope of this thesis 'large' cross sections are defined as those whose continuous (i.e., without intermediate pillar support) cross sectional excavated area exceeds $60m^2$. 'Composite' cross sections are those which embody more than one 'large' cross section alone, normally used for subway stations and junctions.

Figure 3.1 (adapted from Laue et al., 1978) shows several cross section dispositions used in the Bochum subway. Figure 3.2 illustrates schematically the excavation of a stair passageway under the main railway station of this same city (adapted from Schulze, 1982). An examination of the STUVA[1] statistics, published annually (e.g. Haack, 1982), has shown that the number of large cross section

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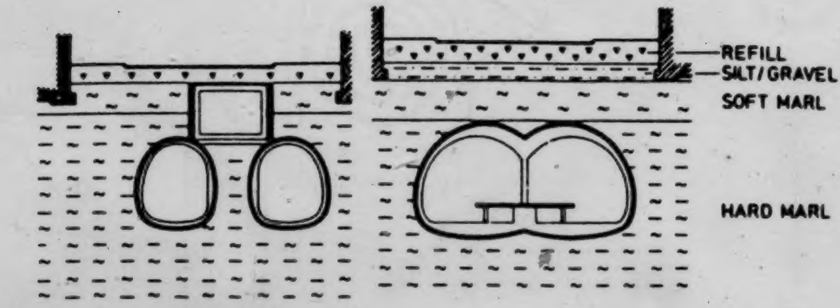
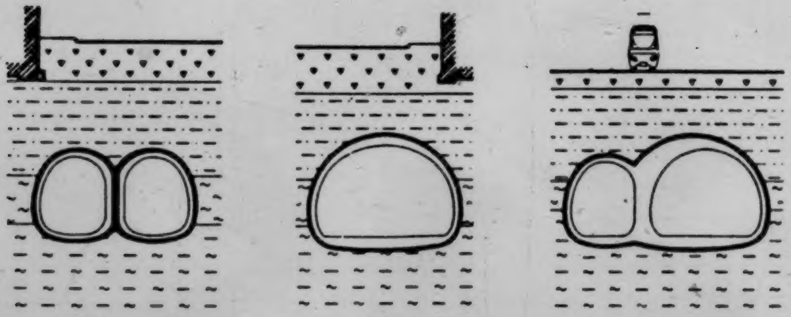
[1] 'German Association for Study of Underground Traffic Works'.


REFILL
SILT/GRAVEL
SOFT MARL
HARD MARL

JUNCTION
(a)

STATION
(b)



(c) (d) (e)

JUNCTION

DOUBLE TRACK TUBE

BIFURCATION

Figure 3.1 Subway tunnel cross sections – Bochum Los A3-A5 (adapted from Laue et al., 1978)

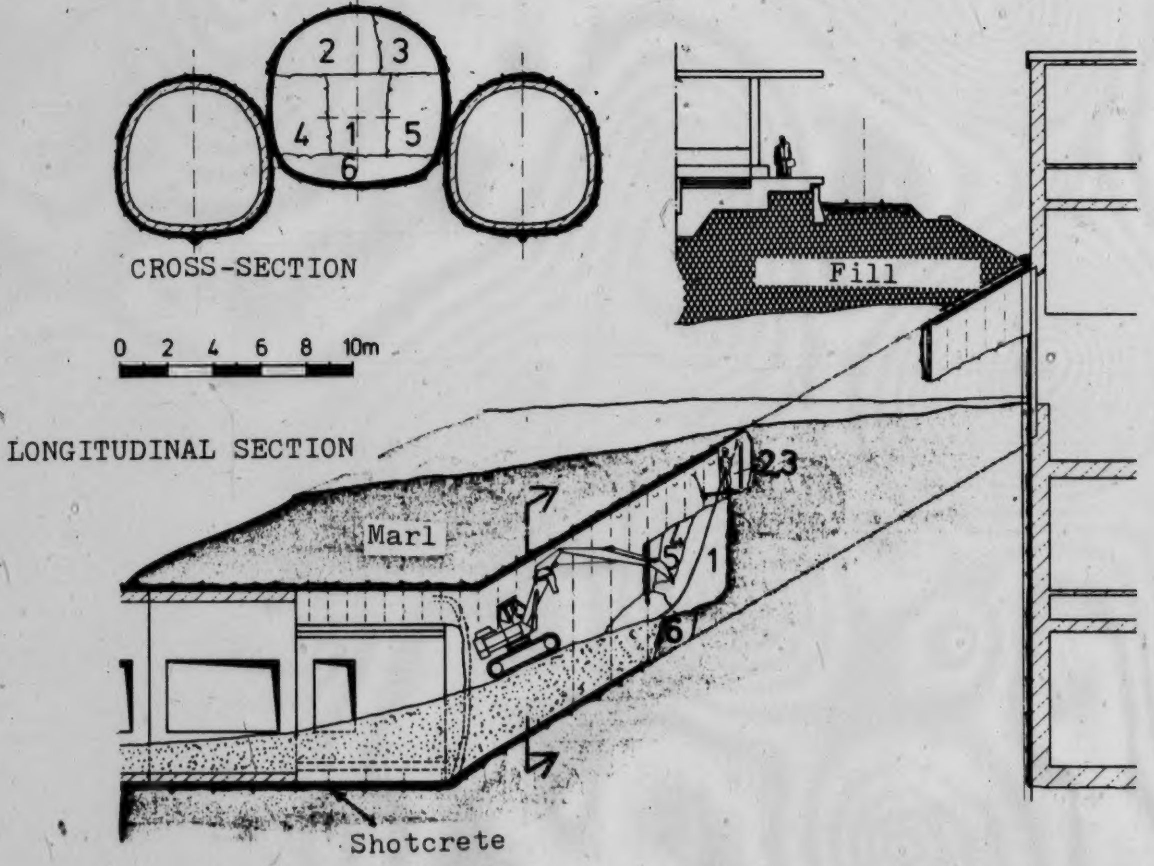

2
3
4
1
5
6
CROSS-SECTION
0 2 4 6 8 10m
LONGITUDINAL SECTION
Fill
Marl
2,3
5
1
6
Shotcrete

Figure 3.2 Excavation of stair passageway under Bochum main railway station (after Schulze, 1982:modified)

works which have been carried out in the last decade using the NATM is appreciable. Composite sections of the order of $200 m^2$ were observed but cross sections ranging from 80 to $130 m^2$ are more frequent. With respect to the ground conditions there is apparently a predominance of stiff cohesive soils.

3.2 Multiple Stage Excavation Schemes

Classical tunnelling methods frequently made use of staged excavation techniques (see for example Széchy, 1973: Chapter 6). Old tunnellers have early verified that the smaller the cross-sectional dimensions of an underground cavity, the more feasible its excavation was, due to either construction or safety reasons.

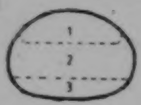
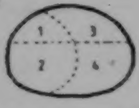
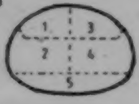
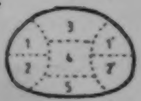
The flexibility and adaptability of the NATM allows an unlimited variety of staged excavation techniques. Those more frequently used for continuous cross-sections (i.e., no intermediate pillars) are presented in Figure 3.3. The classification shown in this figure (i.e., T1, T2, T3 and T4) is being proposed for the first time and is arbitrary. The numbers inside the cross-sections correspond to the sequence of excavation.

Also shown are the approximated percent compositions of the various excavation steps with respect to the total cross-sectional area (A_t) and some information regarding examples of application. In most instances, the value of the modulus of elasticity (E) was the only soil parameter available.

The review of case histories presented in Figure 3.3 is by no means exhaustive, the cases quoted being simply examples found in the literature. It is important to observe that one single step of face advance seldom exceeds 40% of the total cross-sectional area. A trend towards a larger number of stages as the total cross-sectional area increases is also evident. This possibly reflects a concern with tunnel stability and with control of surface settlements.

An important parameter which is related to the ground conditions and to the selection of an appropriate excavation scheme is the time during which the heading is left unsupported just after advancing the face and before placement of the full shotcrete lining. As seen in the preceding chapter, adequate ground control requires that this time should be reduced. Different types of staged excavation would be associated with differences in ground nature, reflecting perhaps a better adaptability of one or another scheme to a specific terrain.

A literature review has shown that the scheme T4 (with side galleries) is the most commonly used. It is apparent that preference is given to this scheme wherever geotechnically problematic formations are present (e.g. presence of low strength horizons or difficult groundwater control). Plate 3.1 illustrates the use of this excavation scheme in a tunnel with a continuous cross sectional area of about 130m² (Essen - Baulos 18) The side galleries are

TYPE OF STAGED EXCAVATION	% TOTAL AREA (*)	EXAMPLES OF APPLICATION/GROUND TYPE
T1 	1=35% 2=45% 3=20%	BOCHUM (Jagsch et al.,1974): At=70m2; marl+sandstone-E=50-100Mpa(**); excavation by road header with some localized blasting NURNBERG (Bauernfeind et al.,1978): At=65m2; sandstone E=220MPa(**); excavation by road header SAO PAULO (Cruz et al., 1982), At=77m2; stiff silty sandy clay-E=80MPa(**); excavation by backhoe using central support core; completed in August 1984
T2 	1=15% 2=25% 3=25% 4=35%	MUNICH (Krischke e Weber,1981): At=88m2; marl+sands +gravels-E=80-200MPa(**); excavation by hydraulic excavator (backhoe) FRANKFURT (Atrott e Sauer,1983): At=72m2; sandy clay- E=40MPa(**); design stage
T3 	1=17% 2=24% 3=17% 4=24% 5=18%	BOCHUM (Jagsch et al.,1974), At=70m2; sandy marl at tunnel axis level- E=10MPa(**) FRANKFURT (Pacher e Sauer,1979): At=109m2; sandy clay- E=40MPa(**); design stage (this scheme is also used for smaller sections)
T4 	1=11% 1'=11% 2=11% 2'=11% 3=17% 4=20% 5=18%	ESSEN (Stadt Essen,1981), At=130m2; silty marls overlying coal- E=40-70MPa(**); excavation by hydraulic excavator (backhoe) - (***) MUNICH (Krischke e Weber, 1981); At=88m2; marls+ sands+gravels- E=80-200MPa(**); ground water infiltration problems-excavation by hydr. excavator

(*) Approximated values derived from literature review
(**) Estimated at tunnel axis level
(***) Plate 3.1 depicts this tunnel

Figure 3.3 Commonly used multiple stage excavation schemes

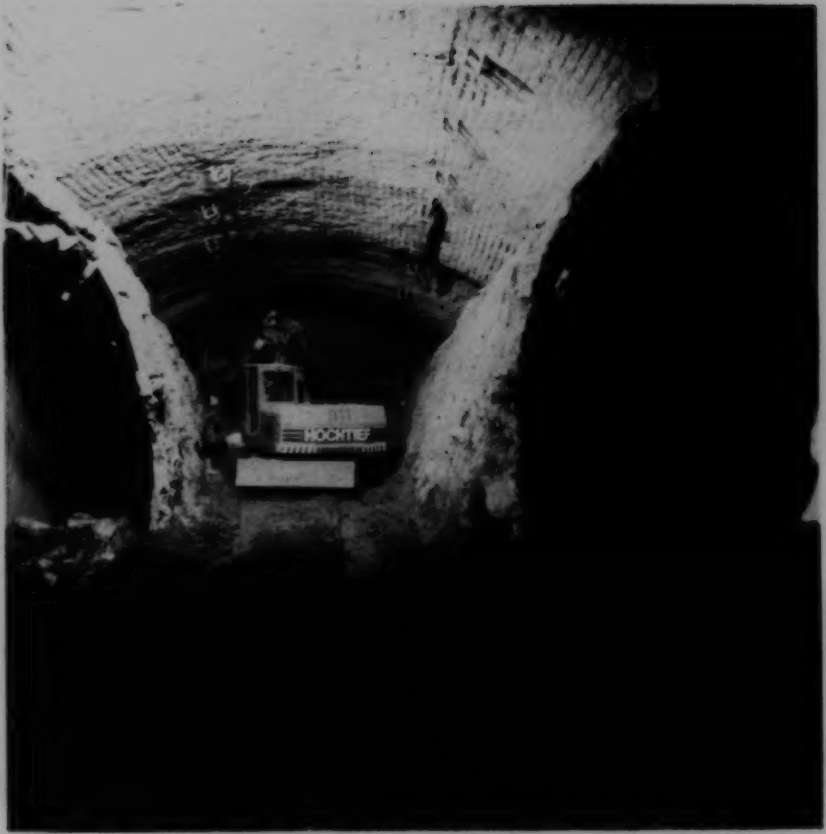


Plate 3.1 Excavation of tunnel in Essen using scheme T4 (photo courtesy of Hochtief A.G.)

normally driven well in advance with respect to the face of the 'main' excavation.

Type T1 (heading – bench – floor) has been applied mostly to stiff soils and soft rocks with better geotechnical properties. Schemes T2 and T3 are variations of T4 and T1 respectively and have been used in soft ground urban tunnels (see Figure 3.3).

3.3 Selection Criteria of Excavation Scheme

Selection of an excavation scheme involves a variety of considerations, some of them of subjective nature and difficult to assess. The topic has, however, been tackled by some authors (e.g. Pacher and Sauer, 1979; Baudendistel, 1979; Krischke and Weber, 1981), a brief introduction being presented herein.

It is recognized that the relative importance of each factor involved may vary from case to case and no rigid rules for selection of an appropriate excavation scheme can be established. The following sections present a review of the main governing factors and are only intented to provide a brief insight into the topic.

3.3.1 Ground Conditions

The geotechnical properties of the ground have a definitive influence upon tunnel face stability and on the magnitude of induced settlements, parameters of vital importance when tunnelling in urban areas. For a given tunnel, an increasing number of excavation stages (and consequent decrease of excavated cross-sectional area for a single step) will normally lead to a safer face advance and smaller settlements.

It is also important to remember that in stiff fissured soils strength decreases with increasing sample size (e.g. Marsland, 1972). Stability of a large underground opening in this type of soil would become even more critical as its size increases. As suggested by Eisenstein and Thomson (1978:344), geotechnical studies in these soils must consider a soil mass rather than extrapolation of results from small samples. In situ tests may be the most appropriate tools in such investigations.

3.3.1.1 Presence of Ground Water

There are cases where ground water table lowering from surface is feasible. In other cases however, it is preferable to drain the soil from inside the tunnel. This requires the use of smaller excavation sections since the presence of ground water may reduce considerably the *stand-up time*[10] of the ground.

In some sections of the Munich underground transit system, perched water tables and some highly permeable horizons showing artesian water pressures were found (Krischke and Weber, 1981). These aspects lead to the selection of type T4 excavation schemes, the side galleries being used to drain the soil mass before the main section was excavated.

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[10] The term stand-up time indicates the time that elapses between the exposure of an area at the tunnel roof and beginning of noticeable movements in the ground above this area (Terzaghi, 1950:193).

By excavating these side drifts in advance, a better examination of the ground conditions is also possible. Information collected normally allows an optimization of subsequent construction stages.

3.3.2 Excavation Equipment

The practitioner normally faces decisions concerning the need for a certain equipment, its availability and related costs. Economically feasible NATM excavation in soft rocks for example, frequently requires the use of 'road headers' (also known as 'part-face tunnelling machines'). On the other hand, excavation in 'softer' materials can normally be carried out solely with the aid of conventional hydraulic excavators.

The problem of selection of an appropriate excavation equipment has been tackled by several authors (e.g. Hammer, 1978; Lessmann, 1980; Clough, 1981). Good, broad reviews of available machinery are presented in the book 'Tunnelling Technology', published by the Ontario Ministry of Transportation and Communications (OMTC, 1976) and in the paper by Theiner (1983). A complete review of these works however, falls beyond the scope of this thesis.

However, since the choice of excavation scheme for an NATM excavated tunnel with large cross-section appears to be closely related to the equipment alternatives, a very brief summary has been prepared and is presented in Figure 3.4. Excavators of the type 1 were the most frequently used in the case histories surveyed. Type 2 has been used (e.g. Müller, 1978a – Chapter 16:552) but in recent years appears to have been abandoned in favour of type 3.

These road headers are equipped with powerful rotating cutting heads and an individual conveyor belt, allowing high rates of advance. The cutting head is rotated and forced against the excavation face in order to cut the soil in pieces for easier removal. Several manufacturers claim that very small cross sections (of the order of $5m^2$) may be handled by these machines, which seem appropriate for example to excavation schemes of the T4 type, when small size side galleries are required.

3.3.3 Cost Considerations

The overall cost of tunnelling appears to be first a function of the rate of advance and is secondly related to the costs of the support. Focussing the staged excavation problem only, one may assume that preference will be given to schemes leading to high rates of advance and that possibly allow for some support savings.

Generally construction schemes of the type T4 are the slowest. Part of the shotcrete used for support of the side galleries is also lost. The same occurs for type T2 schemes while for T1 and T3 schemes no shotcrete is 'wasted'. No

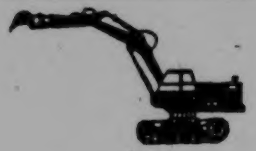
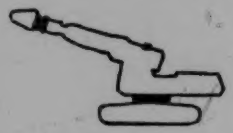
1.HYDRAULIC EXCAVATOR	2.ROAD HEADER	3.ROAD HEADER WITH CONVEYOR BELT
		
APPLICABILITY: -soft to stiff soils ADVANTAGES: -conventional equipment (availability of equipment and personnel) -frequently used in works other than tunnels -relatively low cost DISADVANTAGES: -unfeasible in very small cross-sections -requires additional equipment for muck transport	APPLICABILITY: -stiff soils, soft rocks ADVANTAGES: -higher rates of advance -allows excavation of smaller cross-sections DISADVANTAGES: -requires additional equipment for muck removal and transport -higher costs	APPLICABILITY: -stiff soils, soft rocks ADVANTAGES: -very high rates of advance -practically independent of tunnel size and shape -cross-sections of 4-5 m2 may be excavated DISADVANTAGES: -high initial costs -low availability

Figure 3.4 Type of excavation equipment frequently used in NATM large cross-sectional projects

specific study about these cost comparisons has been found in the literature. The frequent use of scheme T4 suggests that wasting shotcrete and relatively slow excavation progress are minor concerns in urban areas, where an economy in the support may yield other more important costs resulting from damage caused by surface settlements.

3.3.4 Local Traditions

A recognized factor which has an influence on the selected staged excavation system is the 'tradition' factor. Methods which have been successfully used in the past normally tend to gain an acceptance which assures their continuity. These traditions can also be carried over to other places by consultants or contractors.

It is important to recognize the role played by this factor. Injudicious use of traditional methods may appear practical but may also have the effect of stultifying progress in the development of more positive ways of tackling tunnelling problems.

3.3.5 Extent of Ground Movements

In urban areas, limiting settlements which may cause structural damage to surface and subsurface utilities is a major concern. In general, not only the magnitude of the surface settlement but also the slope of the surface settlement depression caused by tunnelling are the prime factors to be considered.

It is apparent that for the same tunnel cross–section, increasing the total number of excavation stages (with immediate application of shotcrete forming a load bearing ring) will reduce the amount of surface settlements. In the Munich tunnels, a 'flat' (i.e., with little distortion) surface settlement depression was sought by means of staged excavations mostly of the T4 type (Krischke and Weber, 1981).

The relative amount of settlements provoked by different schemes could not be assessed through the review of case histories due to absence of settlement performance data for comparison studies. Finite element analyses presented in Chapter 4 attempt to shed some light on this subject with respect to ground conditions found in the city of Edmonton.

3.4 Selected Case Histories

The number of case histories of NATM large cross–sectional tunnels is large, as shown for example by the statistics referred to in Section 3.1. Literature is also available on a number of these cases, but very few have an adequate documentation which could be presented herein.

Two case histories have been selected and are briefly presented in the following sections. They contain particular features that were considered of interest to the present study.

3.4.1 BOCHUM - Baulos A2

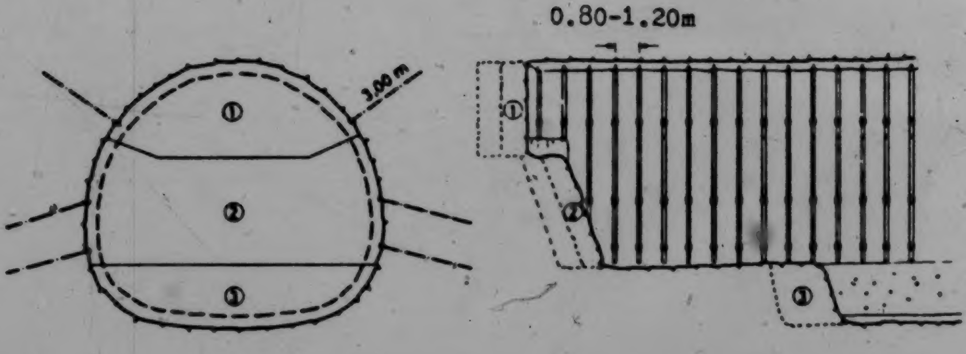
The execution of this double track subway tunnel has been described by Jagsch et al. (1974) and by Hofmann (1976). It represents one of the first NATM cases with large cross-sectional area excavated in urban environment.

The cross-sectional area was about 70m², the soil cover varying between 5.5 and 13m over a length of 300m. Typical ground conditions were represented by marls and locally by sandstones, with water level below the tunnel invert.

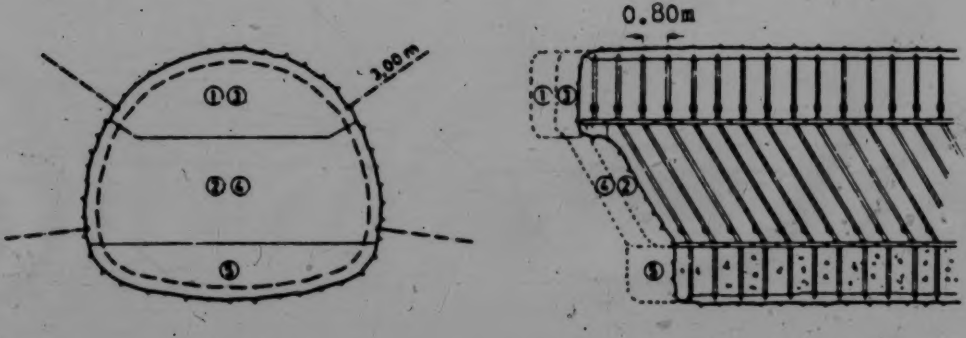
The major part of the contract section was excavated using the type T1 staged excavation scheme, since in these areas ground geotechnical properties were fairly good (E =50 to 100MPa, c'=64 to 320kPa, ϕ'=25°). A road header was used for excavation, combined with some localized blasting of very hard horizons. The invert was closed 9 to 13m behind the face (see Figure 3.5a).

In a short segment underlying an important railway, the ground presented poorer geotechnical properties (horizons with E=10MPa, c'=10kPa, ϕ'=25°). A change to the staged excavation system T3, aimed at minimizing surface settlements, was then carried out. Also, it was decided to reduce to 2.5m the distance behind the face at which the invert was closed. This was achieved by altering the layout and spacing of the steel sets. They were placed in a inclined manner towards the face (Figure 3.5b).

The maximum surface settlements along this critical segment were kept below 20mm, much smaller than those


0.80-1.20m
3.00m
①
②
③
①
②
③

NORMAL EXCAVATION SCHEME


0.80m
3.00m
①③
②④
⑤
①③
④②
⑤

EXCAVATION SCHEME BELOW RAILWAY

Figure 3.5 Excavation schemes used in Bochum - Baulos A2 (after Jagsch et al., 1974:modified)

observed in sections where the scheme T1 was used (Müller and Spaun, 1977).

3.4.2 MUNICH - Line 8.1

It is perhaps in Munich that the most remarkable examples of large cross-section tunnels excavated by the NATM can be found. The case history described herein was reported by Krischke and Weber (1981), representing a double track tunnel (A_t=88m^2) driven trough tertiary deposits [11].

The ground water conditions were particularly unfavourable due to the existence of some pervious sand layers under artesian pressure and several perched water tables. These problems led to the use of a vacuum system of water table lowering. Additional drainage was provided through the side galleries (scheme T4), which were excavated well in advance. The equipment used consisted of hydraulic excavators (backhoes) and pneumatic hammers.

In a length of 290m, three different types of staged excavation were applied, according to the ground geotechnical conditions, basically depending on the thickness of less permeable material existing above crown. The rate of advance was highly controlled by the geotechnical factors, as shown in Figure 3.6.

The increase in the rate of advance in Section 2 when compared to Section 1 resulted initially from an improvement

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[11] Typical properties of the Munich soils are presented in Appendix A, in connection with the description of the Los 8.1-16 case history.

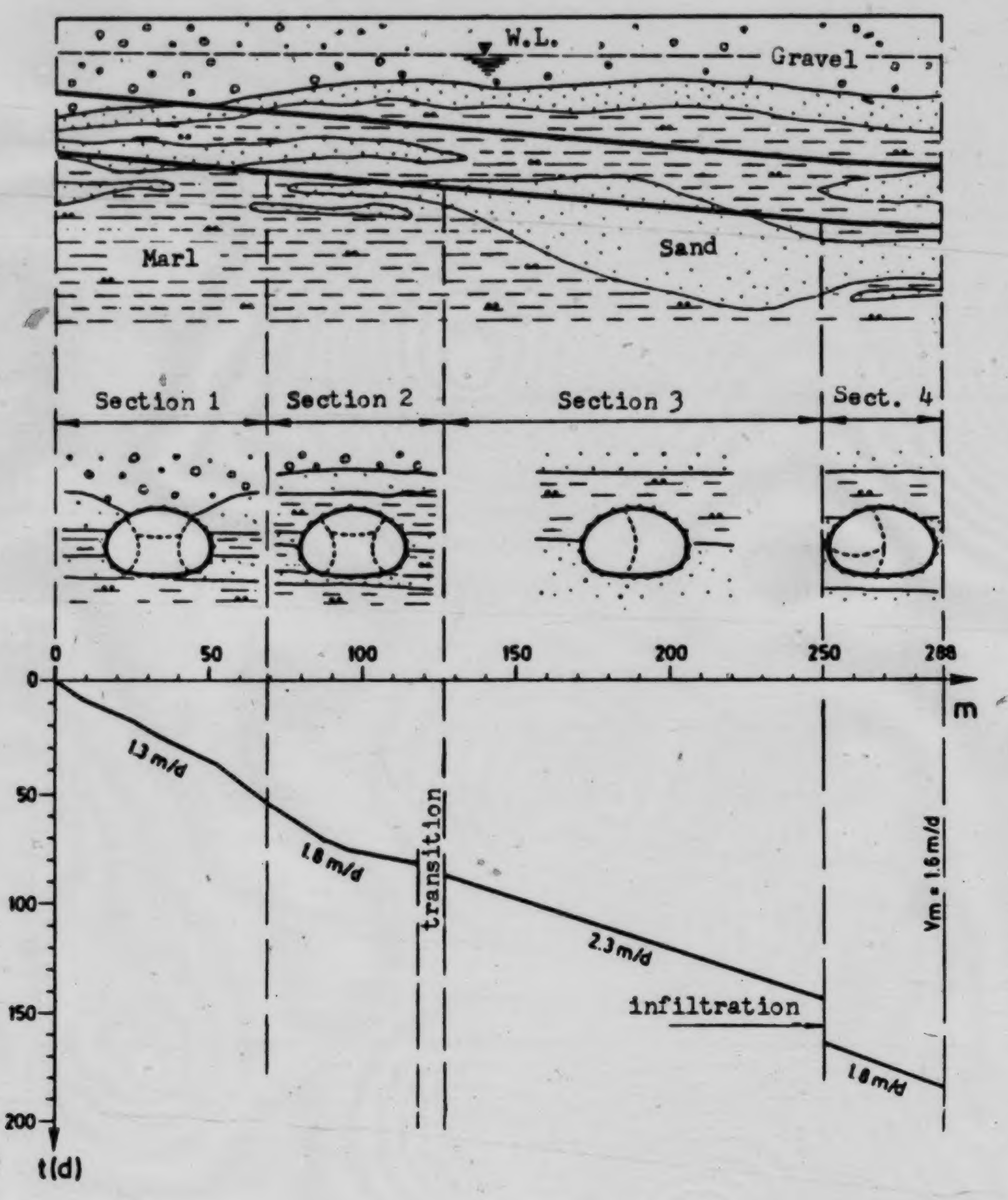
W.L.
Gravel
Marl
Sand
Section 1
Section 2
Section 3
Sect. 4
0
50
100
150
200
250
288
m
0
50
100
150
200
t(d)
1.3 m/d
1.8 m/d
transition
2.3 m/d
infiltration
1.8 m/d
Vm = 1.6 m/d

Figure 3.6 Schemes of excavation used in a tunnel in Munich – A_t=88m² (after Krischke and Weber, 1981:modified)

of the hydrogeological conditions. Recognition of existence of a fairly thick cover of impervious marl in Section 3 allowed the change of construction scheme from type T4 to T2. An increase of the rate of advance was immediately achieved.

Difficulties associated with drainage of a sand layer in Section 4 created the requirement of an elevation of the invert of the side gallery. A temporary shotcreted invert was created. Excavation of this now smaller side drift provided means for solving the drainage problem.

A comparison between the four sections in terms of surface settlements is not possible because data were not available. Diagrams provided by Krischke and Weber (1981:121) suggest that the maximum surface settlements due only to tunnel excavation were of the order of 20 to 50mm.

Plates 3.2 and 3.3 illustrate tunnel excavations in Munich using the T4 and T2 schemes. Plate 3.4 shows a transition from construction scheme T4 to T2[12].

3.5 Comments and Conclusions

The current state of-the-art of soft ground tunnelling in urban areas includes tunnels of large cross-sectional areas. The NATM appears to be the most appropriate tool for excavation of such works. Tunnels with 80 to 130m^2 of cross-sectional area and no intermediate pillars for support

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[12] It is possible that these photographs do not correspond exactly to the case reported by Krischke and Weber (op.cit.).

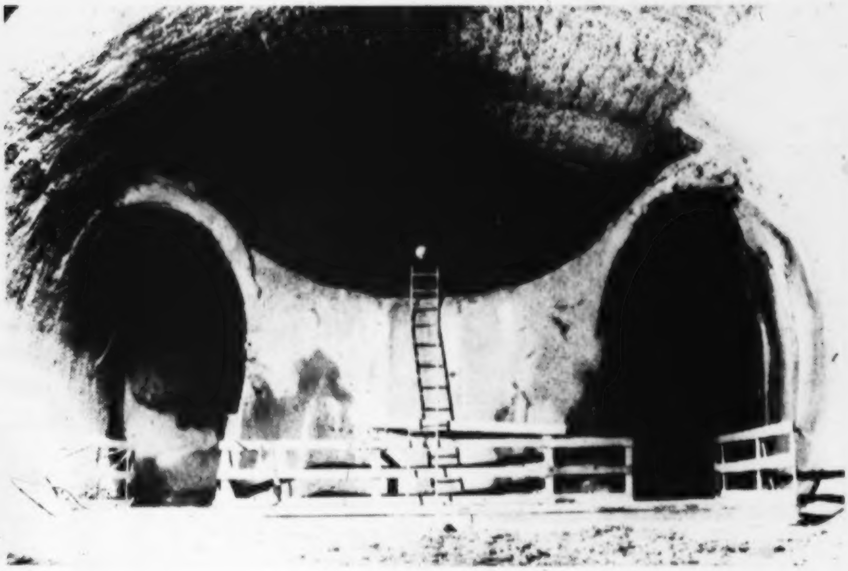


Plate 3.2 Tunnel excavation using scheme T4

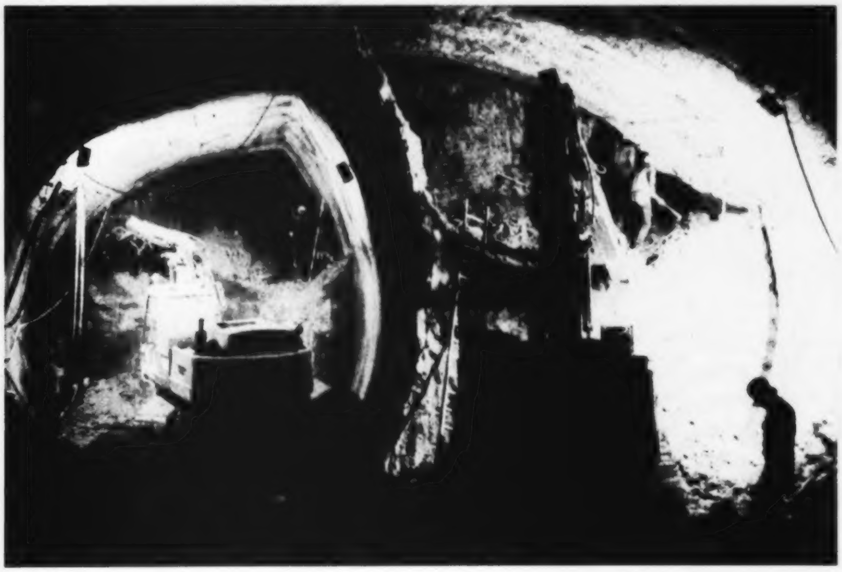


Plate 3.3 Tunnel excavation using scheme T3 (both photos reproduced from 'U-Bahn Linie 8.1', with permission of the U-Bahn Referat – München)

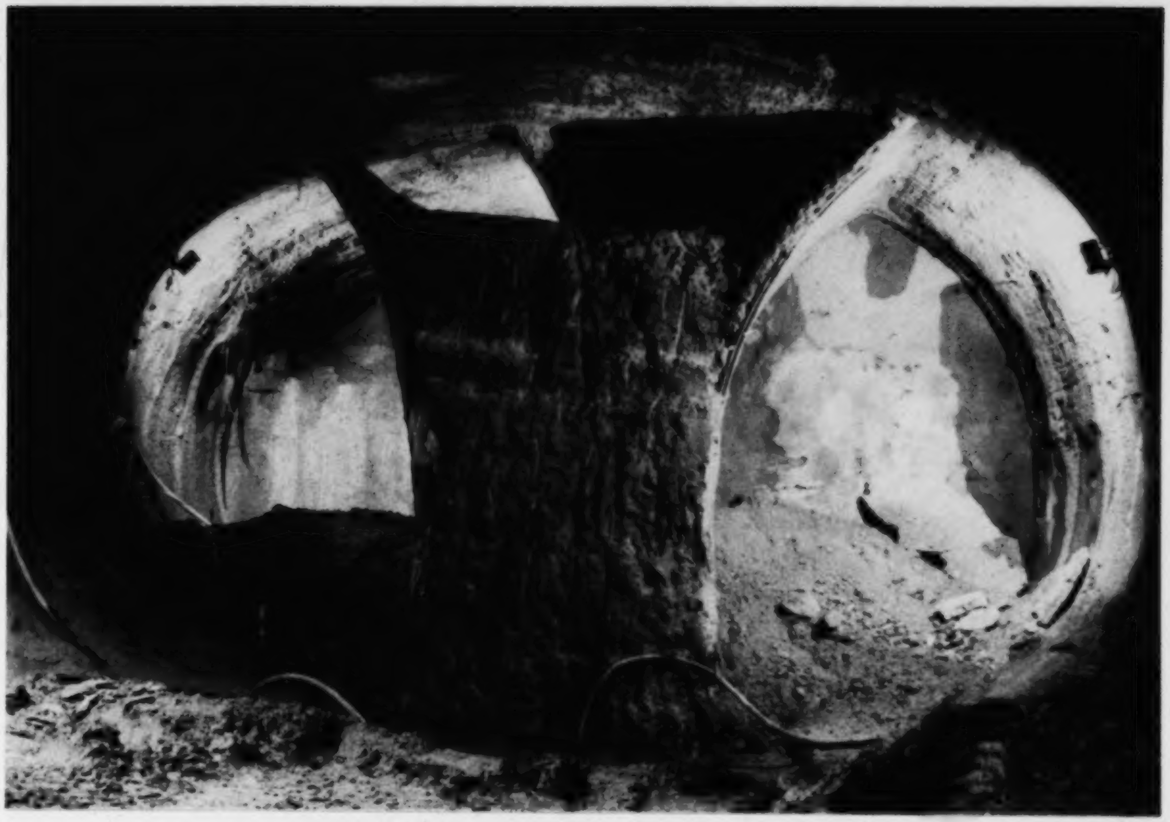


Plate 3.4 Transition from scheme T4 to scheme T2 (reproduced from 'U-Bahn Linie 8.1', with permission of the U-Bahn Referat - München)

have been constructed in a variety of ground conditions. Composite sections of up to 200m^2 of cross-sectional area have also been excavated (e.g. Babendererde, 1980b).

The most efficient way to control ground deformations and maintain face stability in such works is the procedure termed 'staged excavation'. Although an unlimited number of staged excavation schemes by the NATM may be envisaged, there appears to be a tendency to rely on some typical layouts which were reviewed and classified in Section 3.2. Selection criteria were examined but no rigid rules can be established. The most appropriate scheme may then vary from case to case. A frequent scheme used in geotechnically unfavourable grounds is the type T4, where initial advance makes use of side galleries.

The flexibility of the NATM allows fairly easy changes in the construction scheme within a single tunnel as shown in Sections 3.4.1 and 3.4.2. A way to take advantage of this attribute would perhaps be achieved by starting the job with a 'safer' construction scheme (say type T4), changing this scheme to a less complex or rigid one as excavation proceeds and confidence is gained. Adequate site investigations and field instrumentation would clearly play an important role in this 'observational approach' to tunnelling.

Examination of some case histories suggests that the single most important construction factor is the need to close the shotcrete ring in the invert as close as possible to the tunnel face and as soon as possible after excavation of one round. In some instances temporary ring closure may be required before executing the final shotcrete invert.

Design procedures for large cross-section tunnels, which include lining design and prediction of surface settlements, appear to be all closely associated with numerical techniques like the Finite Element Method - FEM (e.g. Swoboda and Laabmayr, 1978; Brem, 1982). Although design diagrams for certain large cross-section shapes already exist (Pierau, 1981, 1982), the simplifyed hypotheses impede its generalized use in design.

An insight into ground behaviour during construction of large cross-section tunnels is provided in Chapter 4, where schemes of staged excavation T1 and T4 are analysed by the two-dimensional FEM.

4. TWO-DIMENSIONAL FINITE ELEMENT ANALYSES

4.1 Introduction

4.1.1 General

This chapter is concerned with applications of the Finite Element Method (FEM) to the analysis of shallow soft ground tunnels. Although the study is mainly devoted to evaluating the capability of the FEM for evaluating NATM performance, some of the concepts involved are more general and may be applicable to simulation of other tunnelling methods.

The theoretical background of the FEM has been described by several authors (e.g. Bathe, 1982) and will not be reviewed herein. Instead, the contents of this chapter will be aimed at describing uses of the method to the analysis of shallow tunnels from a practical point of view. Particular attention is given to specific features of the program ADINA (Bathe, 1978).

4.1.2 Need for Finite Element Analyses

The FEM is specially useful for analysing problems which are not amenable to simple closed form solutions. Due to the presence of the surface boundary, the shallow tunnel problem cannot be rigorously solved by simple elastic solutions such as the traditional Kirsch's equations presented in textbooks (e.g. Hoek and Brown, 1980:103) nor by the elasto-plastic solutions derived for example by Deere et al. (1969). These solutions, as well as procedures such as the 'Convergence-Confinement Method' (Brown et al., 1983) make the implicit assumption that the vertical stress is constant throughout the opening's region of influence, whereas in a shallow tunnel the increase of in-situ stresses with depth has a significant influence on its behaviour (Schmidt, 1970; Mohraz et al., 1975).

An elastic solution which considers the presence of the surface boundary was developed by Mindlin (1939). However, Mindlin's solution does not enable direct calculation of displacements and thus cannot be used for estimating settlements above shallow tunnels. Numerical tools like the FEM are therefore the only suitable procedure to calculate displacements around shallow openings, even in the elastic case. Apart from allowing inclusion of geometrical factors like the presence of the surface boundary, features like geologic discontinuities, non-linear material behaviour and variations of material properties in space and time may also be considered.

4.1.3 Finite Element Program

All analyses presented herein employed the general purpose program ADINA (Automatic Dynamic Incremental Nonlinear Analysis). Although the program was clearly developed from the perspective of a structural or mechanical engineer, it has many attractive features for geotechnical engineering applications, which have been reviewed by Evgin and Morgenstern (1983). Other advantages of this program when compared to more conventional codes are pointed out by several authors (e.g. Dysli and Fontana, 1982; Simmons, 1981:Chapter 4).

Most of the work described herein was carried out using the 1978 version of the program in its original form. Some of the two-dimensional analyses, however, made use of a modified version currently under development (Evgin and Morgenstern, op.cit.). All the program features were evaluated using the descriptions in the program user's manual (Bathe, 1978) and two other publications (Bathe, 1977; Bathe, 1982). Pre- and post-processing were accomplished by small service programs developed either by the Author or by fellow graduate students.

The applications reported herein represent the initial attempts to use ADINA for analysis of shallow tunnels at the University of Alberta. Therefore, information regarding aspects of tunnel excavation simulation which are essential for interpreting the present results are included in the following sections. Some additional information about the program is presented in Appendix D.

4.2 Modelling Criteria

There are some basic requirements that must be fulfilled before finite element analyses can be expected to predict the actual behaviour of a structure. Following the classic paper by Kulhawy (1974), little has been published on the factors which influence these analyses.

The material presented in the following sections is aimed at analysing some of these factors, which have been termed 'modelling criteria' by Kulhawy (op. cit.). The study is directed towards the specific cases analysed in Sections 4.4 and 4.5.

4.2.1 Initial Stresses

As stated in Section 4.1.2, the assumption of initial stresses constant with depth is incompatible with near-surface tunnels. Kulhawy (1975) has carried out a series of linear elastic analyses of tunnels in rock, running equivalent problems with initial stresses constant and varying with depth. The results were made dimensionless with respect to Young's modulus, tunnel radius and initial maximum principal stress.

Kulhawy's study shows that the stresses close to the opening are little affected by variations in the initial gravity stress conditions. However, at more than about one radius, the stress distribution is substantially affected by the assumption made. Based on these results, Kulhawy suggests that gravity stress variation should be included in the analysis of openings shallower than 500ft (150m).

Urban tunnels are normally located at depths shallower than 150m. Furthermore, prediction of surface settlements requires that the surface be a free boundary. Hence, analysis of these tunnels will always require the inclusion of initial stresses varying with depth. A literature survey (e.g. Kulhawy, 1977; Medeiros, 1979) has shown that for analysis of excavations these stresses can be input in two ways:

1. Gravity loading:

When this procedure is used, the initial stresses are obtained by running the program prior to excavation. Displacements due to this initial step are then obtained and have to be subtracted from those resulting from excavation in order to obtain the actual movements. In the analyses presented herein the initial lateral stresses generated by this procedure are automatically taken as:

$$\sigma_y = \nu/(1-\nu)\ \sigma_z \tag{4.1}$$

where σ_y is the lateral (horizontal) stress, σ_z is the vertical stress and ν is the Poisson's ratio[13]. In the three–dimensional analysis, presented in Chapter 5, the σ_x initial stress is obtained by the same manner. The major drawback in this type of procedure is the impossibility of producing K_0 values independent of Poisson's ratio.

2. Initial stress input:

---

[13] This notation corresponds to that used in the program ADINA

In this procedure, the initial stresses are specified directly at each element or group of elements. Implicit advantages are the fact that the initial displacements do not have to be zeroed and that K_0 values higher than unity may be used. Usually, only the values of the bulk density (γ) and of the lateral earth pressure coefficient (either K or K_0) need to be input. They are introduced either in a total or effective stress capacity, depending upon whether a total or effective stress analysis is to be conducted.

This second approach is more appropriate for problems in geotechnical engineering, which will eventually involve high horizontal stresses. In its original version, ADINA does not have this capability. Implementation of a routine which allows input of initial stresses is currently under development (Evgin, 1983) and was used in some applications in the present study.

4.2.2 Excavation Simulation

Simulation of tunnel construction in ADINA is accomplished by the 'birth-death' option. Since the 'birth-death' option is not explained in detail in any of the references assessed, several test runs were conducted to assess this capability. Three important considerations for understanding of the present work which emerged from these test runs are:

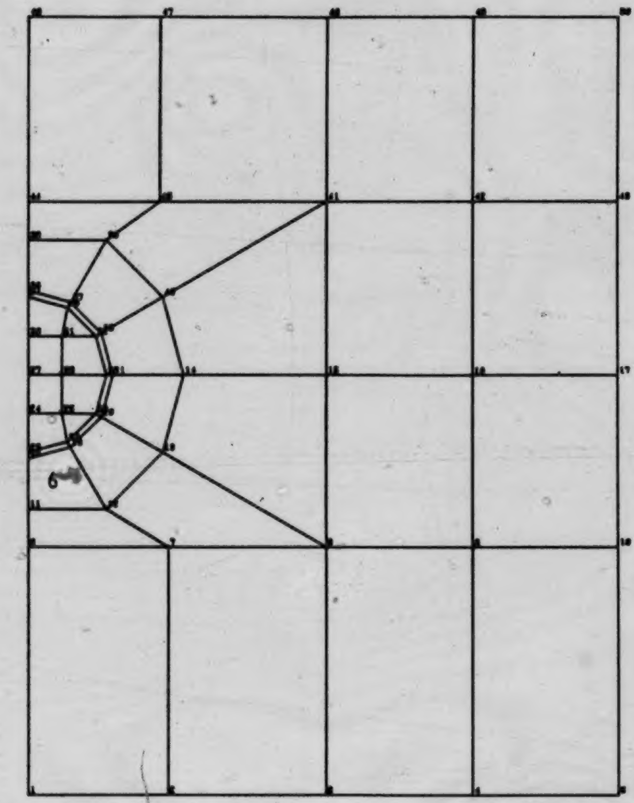
1. Unless gravity stresses are used, the death of excavated elements ensures a stress free excavation boundary.
2. When gravity loads are used as opposed to externaly applied loads, a layer of elements with minimal thickness should be used at the boundary within the area to be excavated otherwise significant errors might be introduced. The reasons for this are explained in the following section.
3. The nodes eliminated during excavation are treated as restrained. This means that they 'freeze' position in subsequent simulation steps until they are eventually reactivated. This aspect is of importance in the procedure adopted to simulate lining erection, as explained in the following paragraphs.

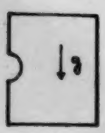
4.2.2.1 Death of Elements

Tunnel construction simulation is easily made possible by using the 'birth-death' option. All elements in the region to be excavated are initially active and at a designated time step, when excavation is to take place, these elements are de-activated, which means that they are not assembled in the stiffness matrix for the remainder of the analysis.

To verify the existence of eventual errors introduced at the tunnel boundary by the birth-death option a circular hole-in-a-plate elastic problem was simulated. In an initial run, the plate is perforated before application of external loads. In the second run, the plate is already stressed when the hole is excavated by using the 'death' option. Since the material is linearly elastic, the stresses generated are independent of the loading history and the stresses in both cases should be the same. Numerical differences generated by the 'death' of the internal elements were not found. A similar problem is depicted in Figure 4.1. In this case, the tunnel is shallow and the in-situ stresses are created by gravity loading. It this analysis the gravity load will be lumped at each elements' nodes. When the element is 'killed', the nodes at the excavation boundary (which remain 'alive') will still be carrying 'residual' loads from the dead elements and therefore the excavated surface will not be stress-free. This problem is minimized by reducing the thickness of the elements at the boundary to a minimum.

In the present study, the elements at the boundary are 'lining' elements (initially specified as soil) which initially have a thickness of 14cm. The results of the test runs shown in Figure 4.1 show that negligible differences are introduced in the stresses at the boundary elements, which may therefore be considered stress free (only the invert stresses are shown, since in the other elements the effect is even less noticeable).



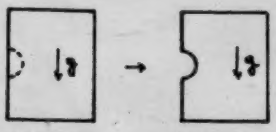


σ in KPa

```
STRESSES FOR EXTERNAL   LOADING CASE

ELEMENT    OUTPUT
 NUMBER  LOCATION       STRESSYY       STRESSZZ       STRESSYZ       STRESSXX

    6
                1      -239.2097      -170.6860       -40.8487      -175.8452
                2      -122.1143       -74.6705       -29.9229       -84.4207
                3      -225.0178      -137.6655       -25.0884      -155.5911
                4       -95.9565       -37.7919       -17.0654       -57.3781
```



```
STRESSES FOR EXCAVATION UNLOADING CASE

ELEMENT    OUTPUT
 NUMBER  LOCATION       STRESSYY       STRESSZZ       STRESSYZ       STRESSXX

    6
                1      -240.4134      -172.7168       -40.9307      -177.2329
                2      -122.7119       -76.1867       -29.9396       -85.3275
                3      -226.0563      -139.4077       -25.0963      -156.7841
                4       -96.3525       -39.0298       -17.0197       -58.0790
```

Figure 4.1 Illustration of errors introduced by the use of a thin layer at the excavated boundary

4.2.2.2 Birth of Elements

The lining elements, initially active as a soil material, are de-activated and later re-activated as shotcrete, except in the analyses presented in Section 4.5, where beam elements were used to represent the lining. Due to the fact that the nodes of 'dead' elements 'freeze' in subsequent steps, the actual thickness of the lining when the elements are re-activated will be smaller than that specified initially. This is due to the fact that nodes at the boundary keep moving inwards during the process of excavation while the nodes in the excavated region are stationary.

In most of the applications reported in this thesis, the actual lining thickness was typically 10 to 13cm. An initial thickness of 14cm was then specified, as a 'safety margin' which would avoid the generation of a lining excessively thin. Maximum deformations effectively obtained from the analyses were of the order of 10mm and therefore it was assumed that the problem reported above would have negligible affect on the results.

4.2.3 Mesh Discretization

Discretization is usually a matter of experience and intuition and therefore it is seldom approached in finite element books and texts. Generally, elements should be smaller where the 'action' is concentrated, i.e., where rapid changes in stresses and strains are expected. However, a very refined mesh will result in higher costs thus the best discretization should be that which yields the required accuracy for the minimum amount of effort (i.e., reduces time for data preparation and output interpretation).

Very few specific studies about the influence of mesh discretization related to shallow tunnels are reported in the literature. Oteo and Sagaseta (1982:653) verified that the density of the finite element mesh has a relatively small influence on the surface settlements (about 10% difference between grids of 68 and 164 elements), but point out that the displacements around the tunnel could be more affected[14].

4.2.4 Boundary Location and Boundary Conditions

Finite element meshes represent a physical approximation of the actual problem and therefore the location of the boundaries with respect to the area of interest (e.g. the tunnel and its surroundings) is expected to influence the results of the analysis. Although fairly extensive studies are reported for deep tunnels (e.g. Kulhawy, 1974), to the author's knowledge little has been published about the shallow tunnel problem. Some attention was therefore given to specification of boundary locations with the aim of optimizing the two-dimensional studies presented in Sections 4.4 and 4.5.

---

[14] In other fields of engineering, the optimum mesh shape is frequently obtained with the aid of interactive computer graphics. Shepard et al. (1979) have used this technique and were able to select near optimum meshes for several practical applications which are worthy of review if optimizing discretization is a concern.

4.2.4.1 Notation

The notation used for the geometric parameters is outlined in Figure 4.2. Several runs were executed and in order to minimize the time required to interpret the results, it was decided to restrict the study to verification of variations of surface and subsurface settlements. Earlier studies published by Pereira and Soares de Almeida (1978) demonstrated that the displacements are very sensitive to the lateral boundary location, while stresses are not.

Table 4.1 lists all executed runs, with the material properties assumed presented in Figure 4.3. The mesh used is shown in Figure 4.4 and is basically the same one used by Negro and Eisenstein (1981). It is important to observe that the boundaries were altered but not the mesh density. All the runs used ADINA's linear elastic isotropic model and were performed as follows:

1. Apply gravity loads to unperforated ground.
2. De-activate core and lining elements (specified initially as soil).

Table 4.1 List of runs for test of boundary conditions

RUN	L/D	Hd/D	EXT.BOUND.	RIG. BASE	MAT.MOD.(*)
1	4	3.2	R(**)	R	2
2	8	3.2	R	R	2
3	8	3.2	R	F	2
4	12	3.2	R	R	2
5	8	2.2	R	R	1
6	8	3.2	R	R	1
7	8	5.2	R	R	1
8	8	7.0	R	R	1
9	8	3.2	R	R	3
10	8	5.2	R	R	3
11	8	3.2	R	R	4
12	8	5.2	R	R	4

(*) MATERIAL MODEL DEFINED IN FIGURE 4.3
(**) F=FIXED / R=ROLLER

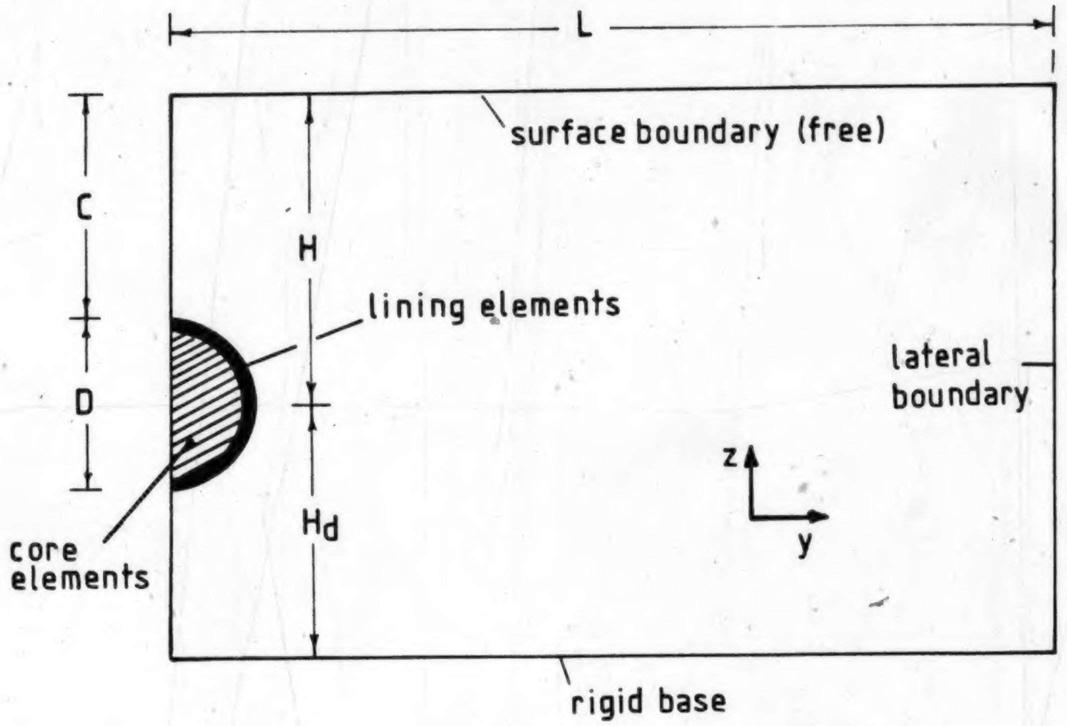

L
C
D
H
H_d
surface boundary (free)
lining elements
core elements
lateral boundary
z
y
rigid base

Figure 4.2 Notation for boundary condition analyses

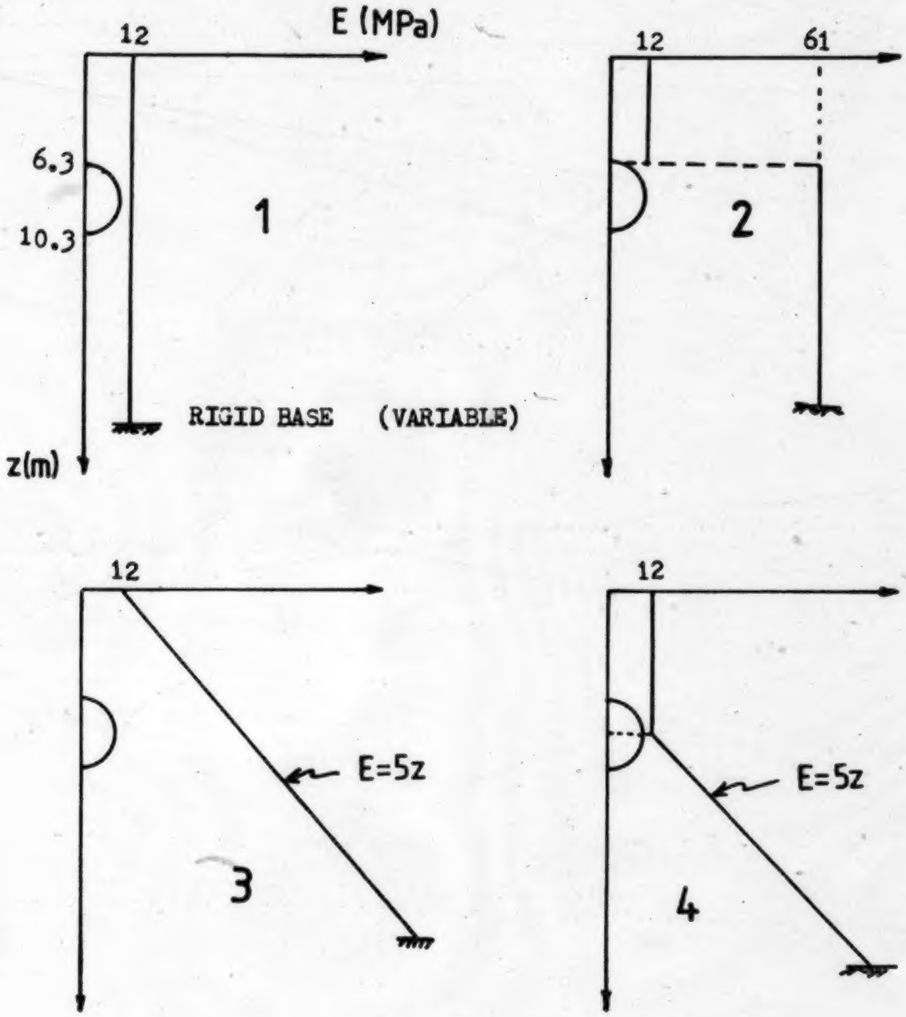

E (MPa)
12
6.3
10.3
1
RIGID BASE (VARIABLE)
z(m)
12
61
2
12
E=5z
3
12
E=5z
4

Figure 4.3 Assumed variations of stiffness with depth

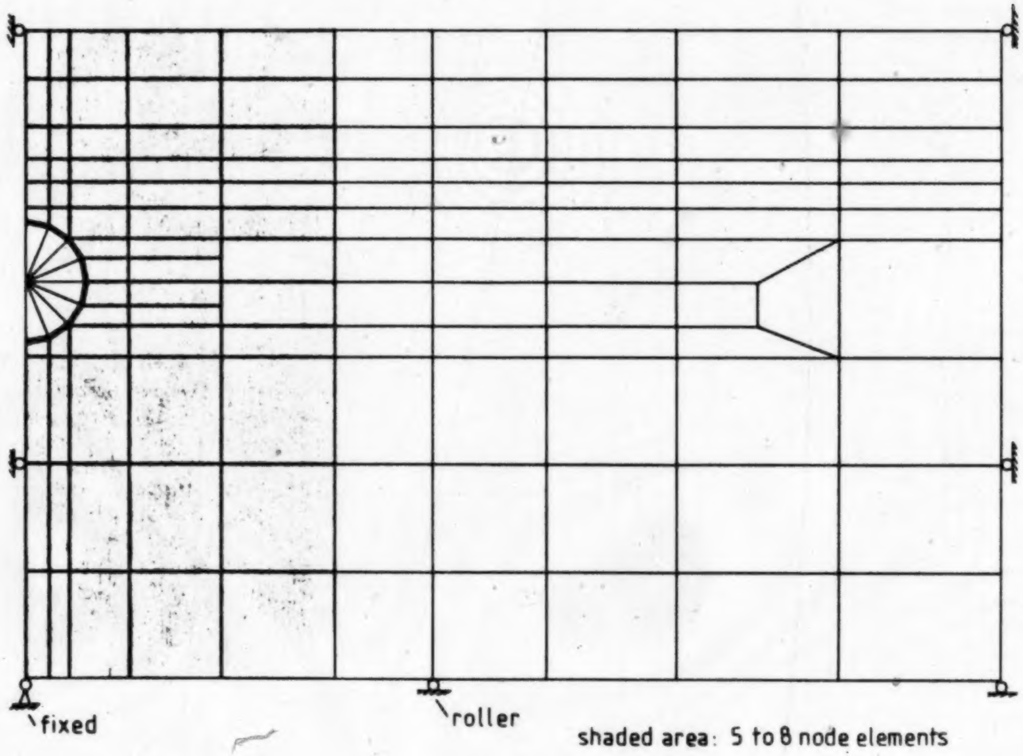

fixed
roller
shaded area: 5 to 8 node elements

Figure 4.4 Two-dimensional mesh

4.2.4.2 Influence of the Distance to the Lateral Boundary

The horizontal distance L (defined in Figure 4.2) has to be chosen so that its length has a minimal influence on the displacement field around the tunnel. This is normally accomplished by placing the lateral boundary at a finite distance L and specifying rollers at the boundary so that the effect on vertical displacements is minimal.

Four different distances of the lateral boundary were investigated, as shown in Figure 4.5. It was verified that the displacements were slightly overestimated for L/D=4 only. For distances of L/D>8 the settlement trough was virtually unaffected[15]. Another point investigated was the influence that the choice of the boundary conditions at the rigid base would have on the settlement. Figure 4.5 shows that neither the maximum settlements nor the shape of the settlement through are affected by changing these boundary conditions.

4.2.4.3 Influence of the Distance to the Rigid Base

An initial insight into the importance of the depth to the rigid base is obtained by a simple approach based on concepts of structural mechanics, as described by Ho

---

[15] Oteo and Sagaseta (1982:653) have observed that the influence of the lateral boundary can be neglected if the ratio L/D is greater than 9 and that for L/D between 5 and 9 the maximum influence in the settlement is less than 10%.

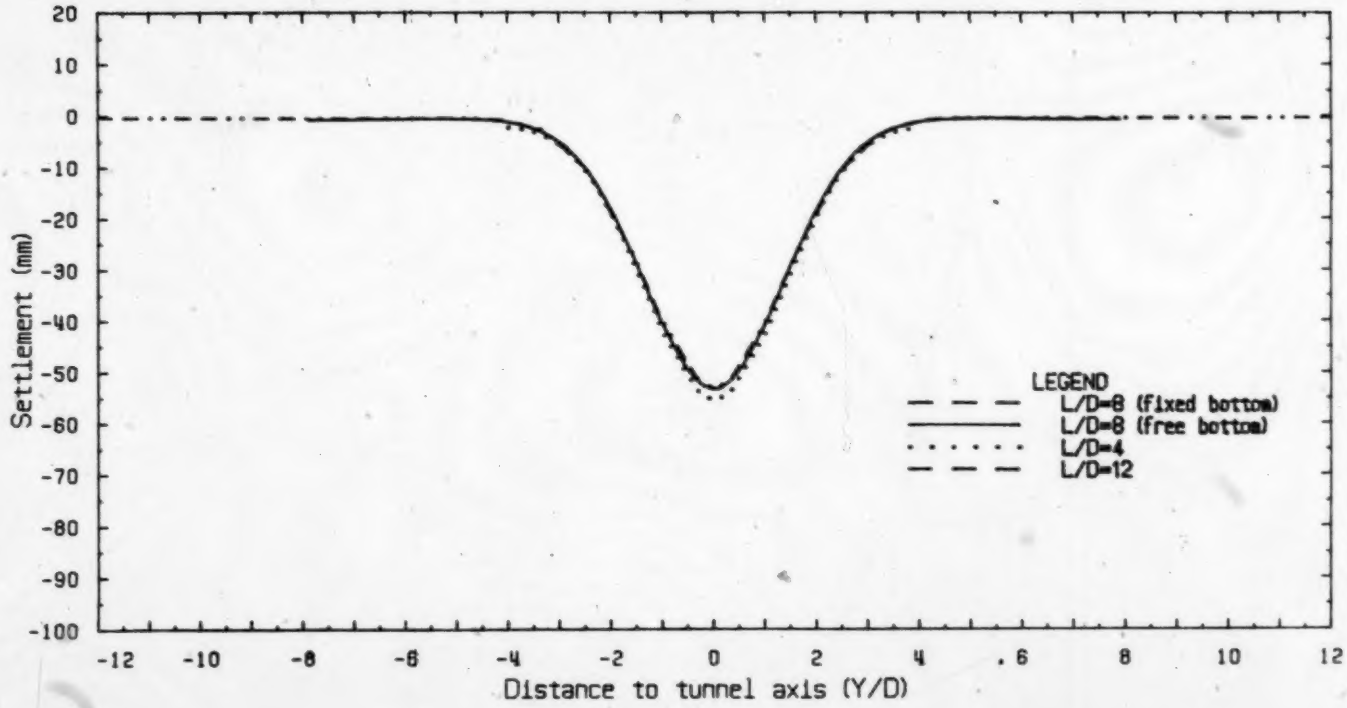


Figure 4.5 Effect of distance to lateral boundary on settlements

(1980) and illustrated in Figure 4.6. The 'settlement components' above and below the tunnel are due to the release of stresses around the opening during excavation simulation (Figure 4.6b). Ho (op.cit.) makes an analogy to the problem of an elastic column subjected to an applied load which is then removed. The displacements at the top of the column may be calculated by integrating the strain along it's length.

In the case of the tunnel, according to the sign convention adopted, downward displacements which occur above crown will be positive, while the heave below the invert will be assigned a negative value. It is clear from Figure 4.6 that the larger the length H_d is with respect to H, the larger the 'negative component' of the vertical displacement will be and a point might be reached where heave will occur.

Several runs were executed to assess the importance of this problem in the analyses presented in Section 4.4. Results of this parametric study are in agreement with the simplified model reported above. This results are depicted in Figure 4.7 and clearly show that in linear elastic analyses with a constant stiffness with depth, the surface settlements predicted will be dependent on the depth of the bottom rigid boundary. This is also the case for the subsurface settlements, which are shown in Figure 4.8. The maximum surface settlement found for a ratio $H_d/D=3.2$ is about 10%

higher than that for $H_d/D=7.2$[11].

It should be noted that the parameter that actually controls the amount of settlements is the ratio H/H_d. This becomes clear if one goes back to Ho's simplified model (Figure 4.6). For $H/H_d=1$ the 'theoretical' surface settlements would be zero, turning into heave for $H/H_d<1$. In actual analyses however, the parameter H will normally be imposed by the problem geometry. The value of H_d might also be determined by the existence of, for example, hard rock below the tunnel. Appropriate judgement would be thus required if analyses with stiffness constant with depth have to be used. This is however unlikely in the case of soils and soft rocks, as commented in the following section.

4.2.4.4 Variation of Stiffness with Depth

The analysis in the preceeding section shows that finite element analyses which assume a constant stiffness with depth cannot provide unique predictions of settlements. These observations are applicable to analyses which make use of linear elastic and elastic-plastic models which contain an initial linear portion of the stress-strain curve. The hyperbolic model used in Section 4.4 takes the variation of stiffness

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[11] Similar comparison using the results presented by Oteo and Sagaseta (1982:654) would result in values about 70% higher. This suggests that other factors, for example the mesh discretization, could play an important role. A comparison was not possible because these authors did not publish their meshes.

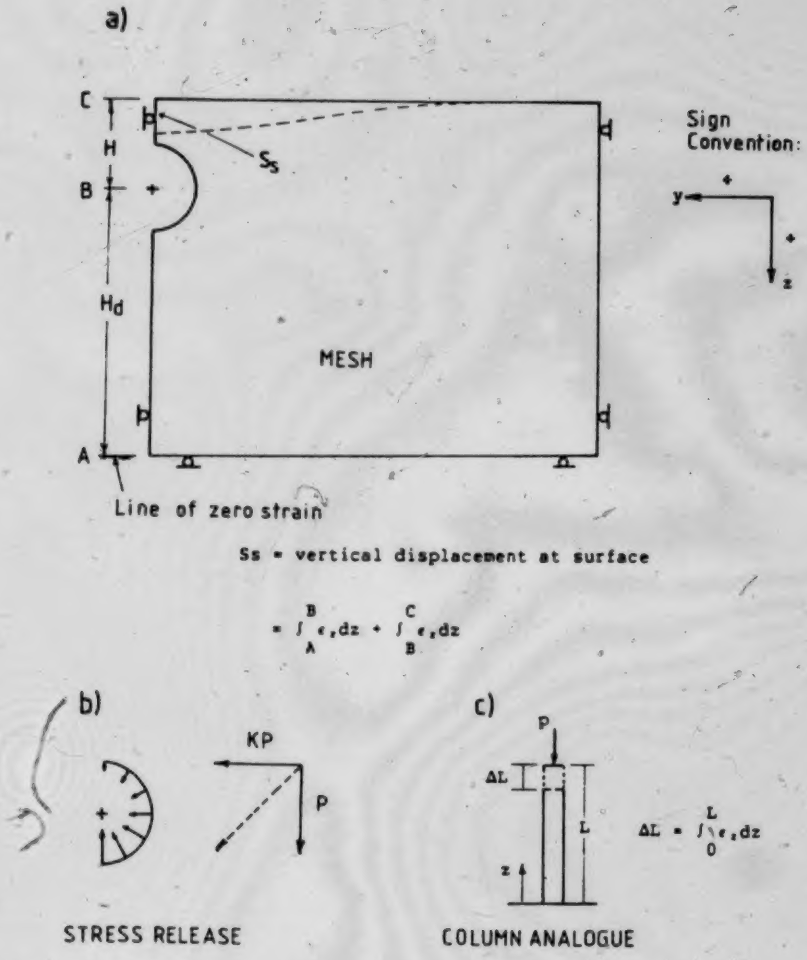
a)
C
H
B
+
H_d
A
Line of zero strain
S_s
MESH
Sign Convention:
y
+
+
z
Ss = vertical displacement at surface
$= \int_A^B \epsilon_z dz + \int_B^C \epsilon_z dz$
b)
KP
P
+
STRESS RELEASE
c)
P
ΔL
L
z
$\Delta L = \int_0^L \epsilon_z dz$
COLUMN ANALOGUE

Figure 4.6 Column analogue to settlement problem in elastic material (after Ho, 1980:modified)

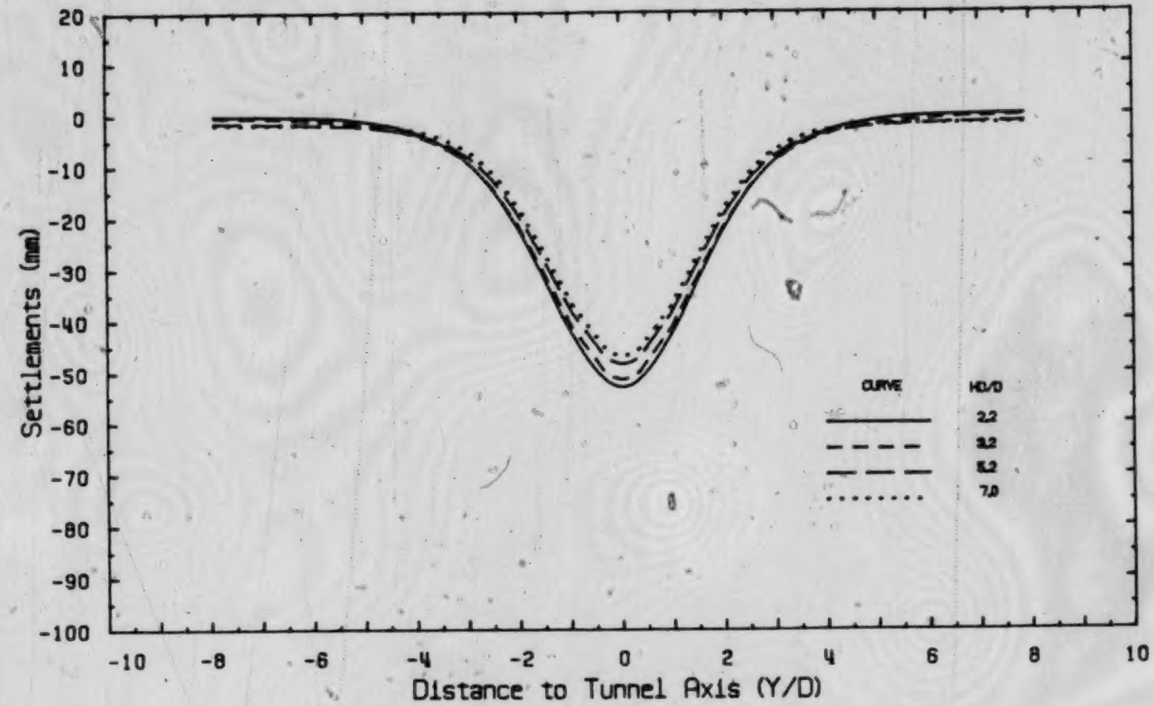

Figure 4.7 Surface settlements as a function of the rigid base position

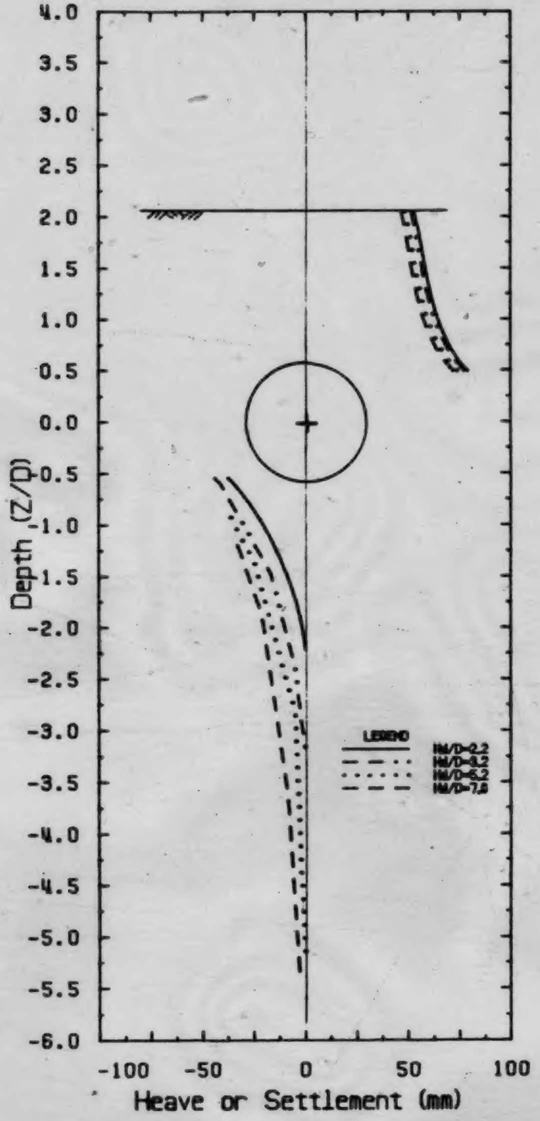

Figure 4.8 Subsurface settlements as a function of the rigid base position

with depth into account, provided the value of n in equation 2 (Figure 4.12) is not zero. The stiffness of soils and soft rocks however, usually varies with depth (Morgenstern, 1975:9) and therefore analyses taking this more realistic assumption into account would be expected to be more accurate. Some assumptions for the variation of stiffness with depth approximately compatible with the field conditions of the ABV tunnel analysed in Section 4.4 were investigated[17]. The results, depicted in Figures 4.9 and 4.10 show that in these cases the magnitude and distribution of the surface and subsurface settlements are practically insensitive to variations of the parameter H_d[18].

Another feature that can be noted is the considerable influence that the assumed variation of stiffness with depth (i.e., material 2, 3 or 4) has on the surface settlements. It is clear that a realistic evaluation of the stiffness with depth will be a matter of primary importance in analyses of shallow tunnels aimed at predicting settlements.

4.2.5 Material Model

Appropriate representation of the soil behaviour around an excavated shallow tunnel is known to be an important issue when accurate predictions are required (e.g.

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[17] These assumptions were shown in Figure 4.3.

[18] Oteo and Sagaseta (1982:654) point out that the operating modulus below the invert should be higher than that above.

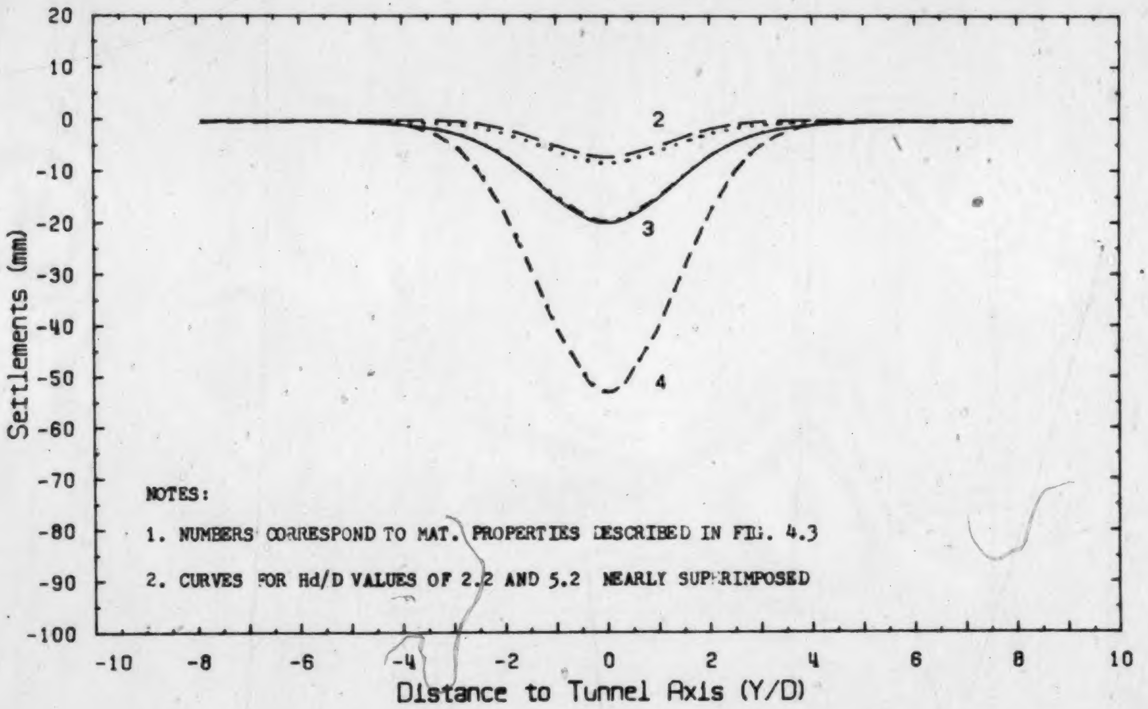


Figure 4.9 Surface settlements as a function of the rigid base position with different assumptions of stiffness variation with depth

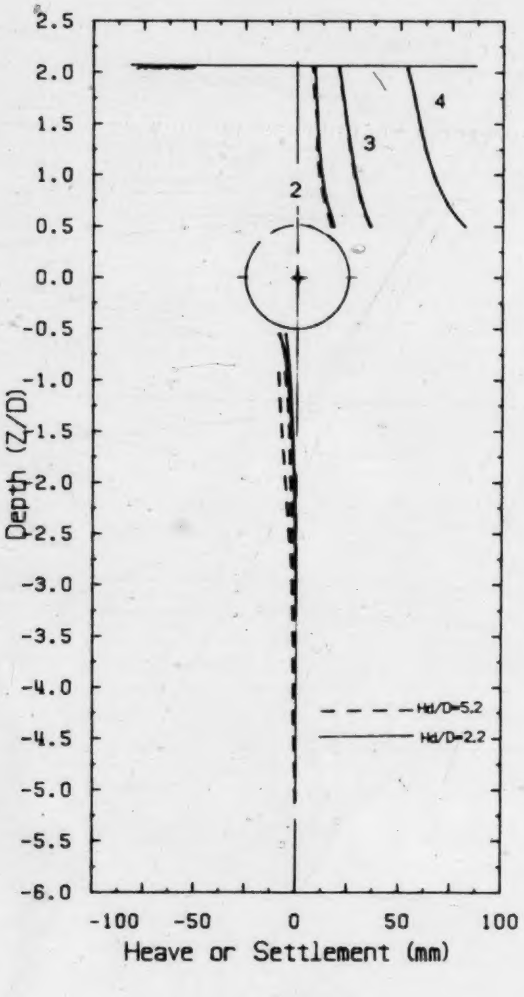


Figure 4.10 Subsurface settlements as a function of the rigid base position with different assumptions of stiffness variation with depth

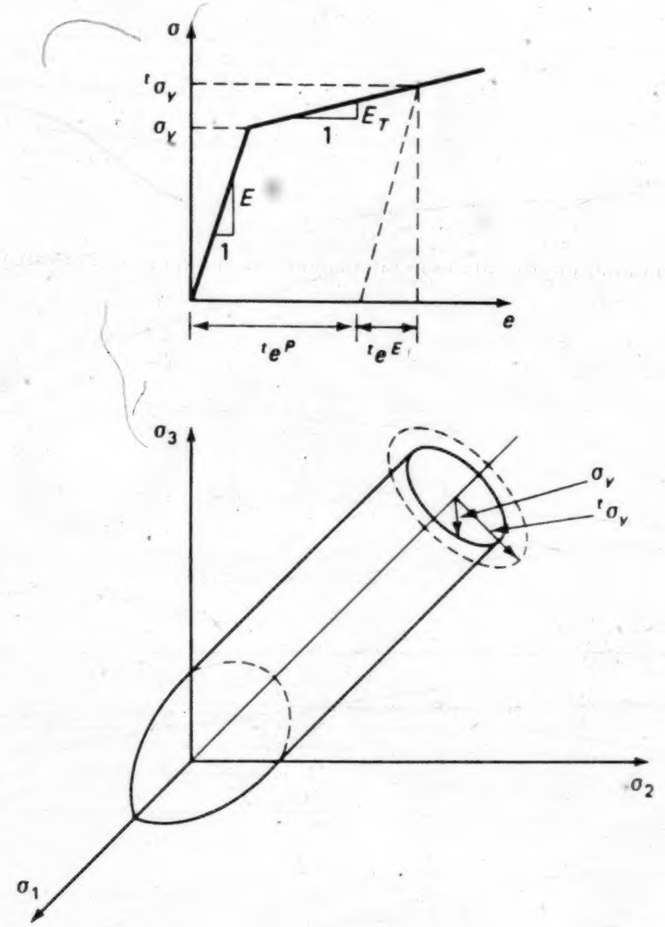
Eisenstein, 1982). The program ADINA offers several choices of material models. All these models are described in the book by Bathe (1982) and will not be reviewed herein.

The two material models available in the original version of the program and used in this thesis are the 'Linear Elastic Isotropic' and the 'Elastic-Plastic with von Mises Yield Criterion and Isotropic Hardening'. The other material model used was the hyperbolic model for soils, introduced at the University of Alberta (Evgin, 1983).

4.2.5.1 von Mises Model

This model is illustrated in Figure 4.11 which summarizes its main properties. The derivation of the equations used to describe the yield function and the calculation of the plastic displacements is fully presented by Bathe (1982:388) and will not be reviewed herein.

The model makes use of the classical von Mises yield criterion used for interpretation of tests on the plastic behaviour of metals. The value of the yield stress (σ_y) is normally taken as the yield stress in simple axial tension tests on steel. If it is assumed that the yield limit in tension is the same as in compression, σ_y may be taken as q_u which is the uniaxial compressive strength of the soil in undrained tests. Successful use of this approach has been reported by Dysli et al. (1979) who used the program ADINA.


σ
${}^t\sigma_y$
σ_y
E_T
1
E
1
e
${}^te^P$
${}^te^E$
σ_3
σ_y
${}^t\sigma_y$
σ_2
σ_1

von Mises Model

(Isotropic Hardening)

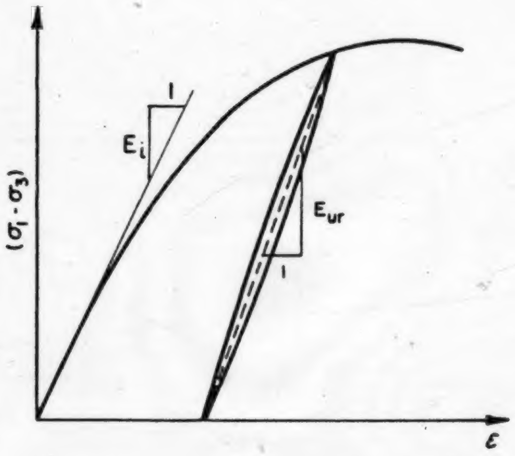
Figure 4.11 Illustration of von Mises Model (after Bathe, 1982:modified)

4.2.5.2 Hyperbolic Model

This is a soil model which was introduced in non-linear incremental stress analysis by Duncan and Chang (1970). The model is based on the assumption that the stress-strain curves of triaxial compression tests for certain soils could be represented by a hyperbola.

The hyperbolic model is illustrated in Figure 4.12, which summarizes the main equations. In non-linear incremental analysis of stresses and deformations, each increment assumes the soil as being piecewise linear elastic, the tangent modulus E_t varying according to variations in the principal stresses. Variation of the stress-strain behaviour with confining pressure is taken into account by equations (2) or (3) in Figure 4.12, depending on whether the soil is being loaded for the first time or is experiencing an unload or reload behaviour.

The hyperbolic model has been used extensively for the analysis of deformations of large structures in a variety of soils. Recent applications on the analysis of soft ground tunnels have shown that use of this model can provide good estimates of surface settlements above such structures (Kawamoto and Okuzono, 1977; Katzenbach and Breth, 1981). Critiques on the original hyperbolic model have been made and were mainly directed towards its inability to take into account volume changes due to changes in shear stress.


$(\sigma_1 - \sigma_3)$
E_i
1
E_{ur}
1
ε

$$E_t = E_i\{1-[R_f(1-\sin\phi)(\sigma_1-\sigma_3)]/[2C\cos\phi+2\sigma_3\sin\phi]\}^2 \qquad (1)$$

$$E_i = Kp_a(\sigma_3/p_a)^n \qquad (2)$$

$$E_{ur} = K_{ur}p_a(\sigma_3/p_a)^n \qquad (3)$$

R_f = failure ratio

K = modulus number

Kur = modulus number for unload-reload

n = modulus exponent

p_a = atmospheric pressure

Figure 4.12 Illustration of hyperbolic model

Also, experience shows that the model is not suitable for the analysis of stresses and movements close to failure. Christian (1982:194) suggests that the hyperbolic model gives reasonably good results for stress levels less than about 75% of failure. Nevertheless, the model can be used to represent the stress-strain behaviour of a wide range of soil types and has the advantage of using parameters which can be readily interpreted in terms of its physical significance.

4.2.5.3 Verification of Hyperbolic Model

The work which is presented herein was carried out with an experimental version of the model, which did not have the feature of equilibrium iterations implemented[19]. This created the necessity of verifying the effect that the number of steps would have had on the results. This was done by simulating a simple triaxial test with different numbers of steps used to apply the incremental axial loads. The results were compared to results of another program (NLCP, from Simmons, 1981) and to hand calculated values[20].

Figure 4.13 summarizes the test configuration and material properties assumed. It should be noted that only one quarter of the triaxial specimen need to be

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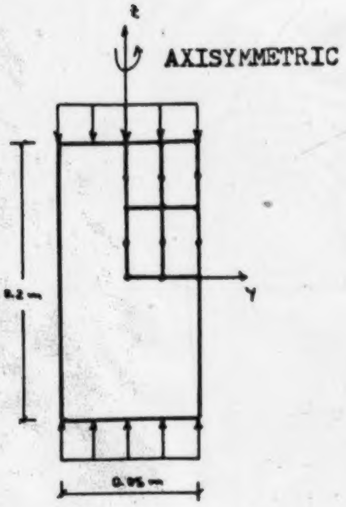
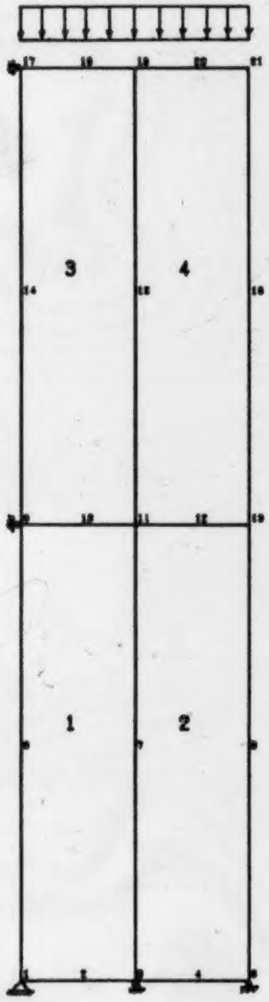
[19] This is a technique used in finite element non-linear analyses which allows for an update of the stiffness withouth the need of applying loads in small increments.

[20] The hand calculations and the results from the NLCP run were provided by Mr. D. Chan.


z
AXISYMMETRIC
Y
0.2 m
0.05 m


17
18
19
20
21
14
15
16
3
4
10
11
12
13
1
2
6
7
8
1
2
3
4
5

MAT. PROPS. (Loose Sand)

$\gamma = 0.0$

$K = 295$

$K_{ur} = 1090$

$n = 0.65$

$c = 0.0$

$\phi = 30.4$

$R_f = 0.9$

$\nu = 0.30$

Figure 4.13 Scheme for tests of hyperbolic model

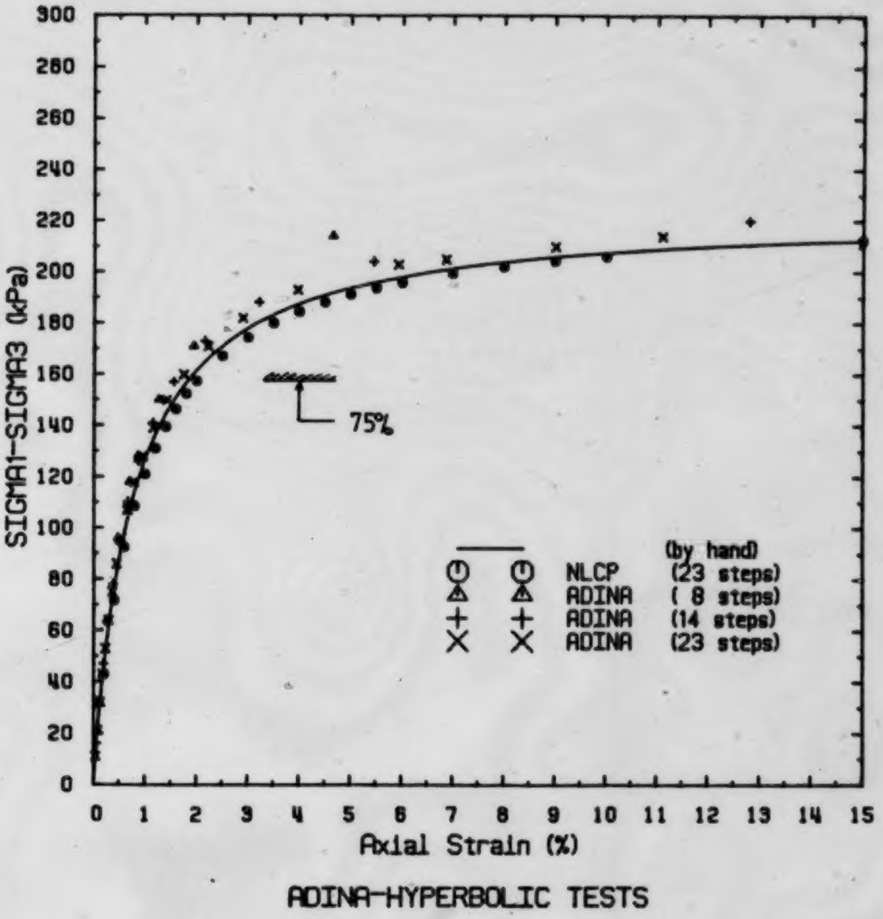


Figure 4.14 Results of tests of the hyperbolic model

represented in the axisymmetric mesh. The confining pressure is introduced by specifying the stress state of each element and only the deviatoric load is added to the sample. Figure 4.14 depicts the results which show that the model produces results comparable to the exact solution, provided a large number of steps is used. However, the results up to 75% of the failure stress are practically insensitive to the number of steps used to apply the load. It was concluded that the number of steps would not be a decisive factor in the problem analysed, which in the numerical analyses did not involve mobilization of shear stresses above the 75% level[21].

4.3 Simulation of NATM Excavation

The advance of an NATM tunnel with the heading and bench procedure is very difficult to simulate in two dimensions. However, the fact that three-dimensional analyses are complex and time consuming, motivated the development of several approximate two-dimensional procedures. Eisenstein (1982) has reviewed some of these techniques. Basically, three general procedures can be distinguished. They are illustrated in Figures 4.15 and 4.16 and briefly described in the following sections.

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[21] As will be seen in the following sections, the stresses and displacements close to the tunnel boundary could not be properly evaluated due to numerical innacuracies. It is possible that in these areas the 75% level was exceeded.

4.3.1 Core Modulus Reduction Procedure

It is known that some movement takes place in the ground mass before the support is placed. This movement will reduce the loads carried by the support according to the concept of ground response curve discussed in Chapter 1. This 'pre-support' behaviour is included in a plane strain analysis using the method advocated by Swoboda (1979)[22].

In this procedure, the ground mass is initially unperforated. Before the tunnel is excavated and the support is placed, the elastic modulus within the 'core' is reduced piecewise to zero (see Figure 4.15a) and the ground moves radially inwards by a certain amount. The support is activated in two steps. Swoboda (op.cit.) sought mainly a more realistic assumption for the lining loads by using this stepwise reduction of the core modulus. Schikora (1982, 1983) has applied similar procedures to the prediction of surface movements in Munich tunnels, with encouraging results.

This type of simulation cannot be carried out with the program ADINA in its current state. Attempts were made to use the 'restart' option in the program to stop processing and change material properties, but it was verified that this could not be accomplished. This is due to the fact that using the restart option, the original material properties are stored for all subsequent calculations. De-activating the core (with initial modulus E) and immediately replacing it with a reduced modulus is possible by specifying two different element groups for the core (the group with reduced modulus initially inactive). However, one element group cannot be activated and de-activated in the same run and therefore the procedure cannot be completed.

---

[22] According to Steiner et al. (1980:98) this procedure was primarily developed for design of tunnels constructed by the 'ramp method' (see Appendix A).

4.3.2 "Progressive Core Removal" Procedure

This technique was introduced by Wanninger and Breth (1978) being aimed at simulating the sequential construction as in the core modulus reduction procedure. The core is discretized in great detail and gradually removed during various excavation steps. Figure 4.15b illustrates this feature including the sequence of lining erection. Katzenbach (1981:46) observed that the technique yields realistic displacement predictions but that the lining loads are meaningless. This is due to the fact that when the shotcrete ring is fully completed, 100% of the stress release has already taken place.

Although this procedure could be easily modelled with ADINA, it was not implemented due to the reasons reported above. It was decided that instead the technique presented in the following section would be attempted.

4.3.3 Gradual Boundary Stress Reversal

This procedure is frequently used in two-dimensional finite element simulation of shield tunnels. It consists

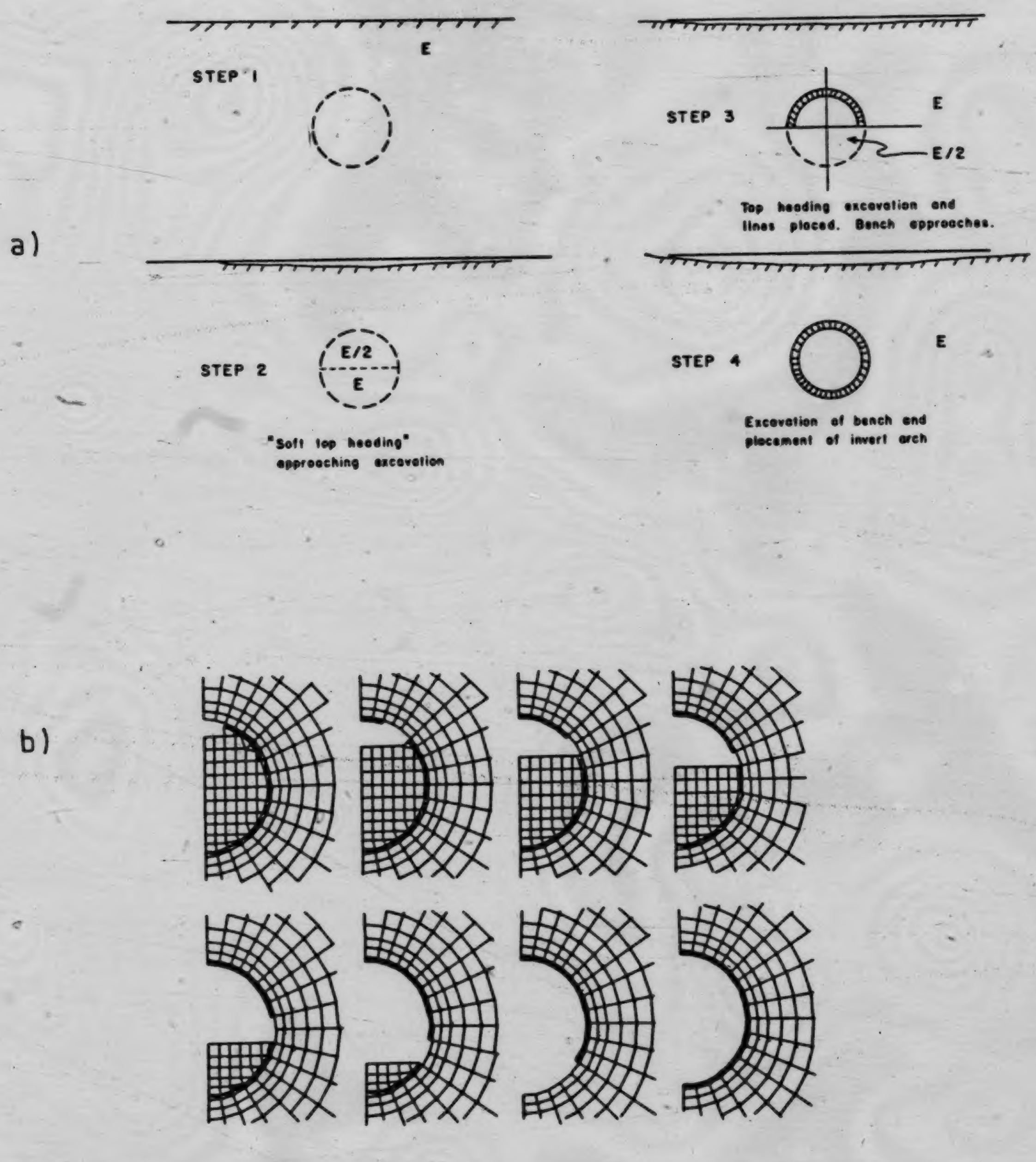

a)
STEP 1
E
STEP 3
E
E/2
Top heading excavation and lines placed. Bench approaches.
STEP 2
E/2
E
"Soft top heading" approaching excavation
STEP 4
E
Excavation of bench and placement of invert arch
b)

Figure 4.15 NATM simulation techniques in 2D (modified after Steiner et al., 1980 and Wanninger, 1979)

basically of excavating the 'core' elements and then applying a gradual release of boundary stresses to simulate tunnel advance.

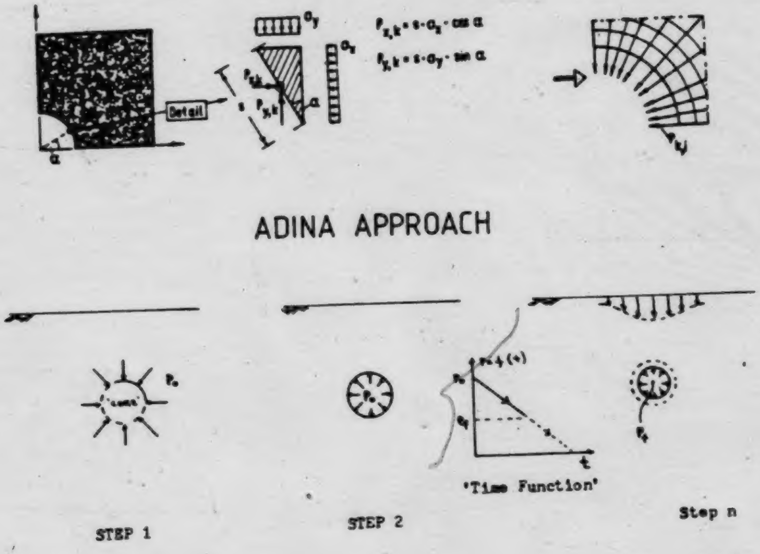
The technique is illustrated in Figure 4.16a. The original in-situ stress at a point at the tunnel boundary is gradually reduced by an amount corresponding to this percentage. At the same time, this point will displace towards the tunnel center by virtue of the loads applied.

Application of this technique to ADINA raises a problem. This is due to the fact that when the core elements are de-activated, the 'birth-death' automatically applies 100% of stress release to the boundary. The way devised to overcome this problem was to implement an internal support pressure to the tunnel at the time of the 'death' of the core element, as shown in Figure 4.16b. This pressure, initially equal but opposite to the in-situ stresses, is gradually released by using ADINA's 'time functions' thus simulating the tunnel advance.

Also illustrated in Figure 4.16b is the percentage of boundary stress release, hereafter termed 'percentage of stress release' or simply %SR. It will be seen in the following section that the value of %SR to be used in the 2D analysis is particularly critical in predictions of settlements.

CONVENTIONAL APPROACH

ADINA APPROACH


σy
σx
P_x,k = s·σx·cos α
P_y,k = s·σy·sin α
Detail
P_x,k
P_y,k
s
α
P_xj
Po
Po
P = f(t)
Po
Pf
t
'Time Function'
Pf
STEP 1
STEP 2
Step n

Percentage of Stress Release - % SR = 1 - Pf/Po

Figure 4.16 Gradual boundary stress reversal techniques

4.4 Back Analysis of São Paulo ABV Tunnel

This case history is described in Appendix A. It was chosen for the present analysis because of the availability of considerable field data and also due to the fact that a similar study had been performed before (Negro and Eisenstein, 1981) so a check of the ADINA procedures could be made.

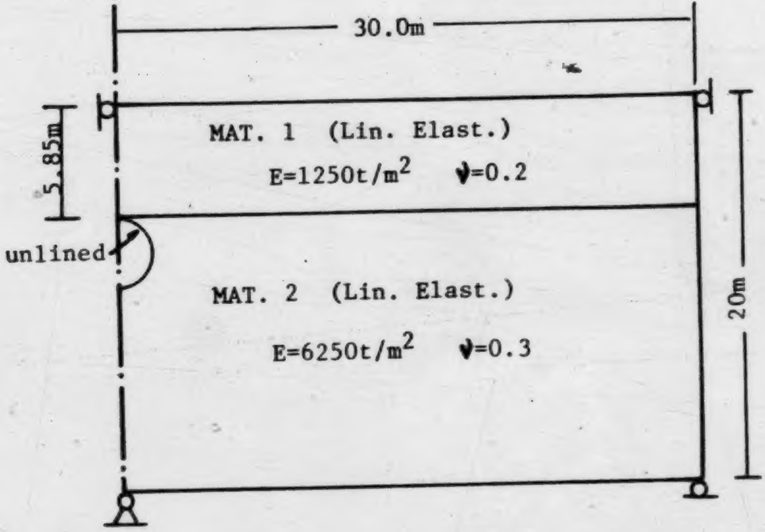
4.4.1 Assumptions

In their back analyses, Negro and Eisenstein (op.cit.) used the properties and boundary conditions outlined in Figure 4.17a. The analysis was carried out in one step only (i.e., 100% of stress release in one step) and no lining was placed. These authors considered the matching of surface settlements as the most important criterion and 'pseudo moduli' were derived for the soils. The use of this approach did not allow matching of the subsurface settlements.

The initial run reported herein was carried out using the same moduli derived in the previous study. Subsequent runs made use of hyperbolic parameters for the variegated soil. These parameters, which are listed in Figure 4.17b, represent undrained values and were derived from earlier studies by Sousa Pinto and Massad (1972), following procedures described by Duncan et al. (1980).

The porous clay is a lateritic soil which is known to present a somewhat erratic behaviour with extreme variations in stiffness properties within the same deposit. Generally,

a) NEGRO AND EISENSTEIN (1981)


30.0m
MAT. 1 (Lin. Elast.)
E=1250t/m² ν=0.2
5.85m
unlined
MAT. 2 (Lin. Elast.)
E=6250t/m² ν=0.3
20m

MATERIAL 1 - POROUS CLAY MATERIAL 2 - VARIEGATED SOIL

b) PRESENT STUDY

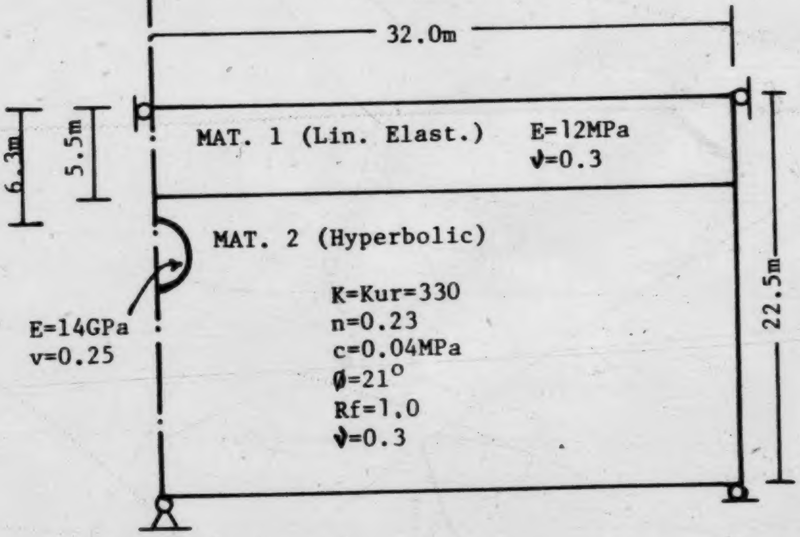

32.0m
6.3m
5.5m
MAT. 1 (Lin. Elast.)
E=12MPa
ν=0.3
MAT. 2 (Hyperbolic)
K=Kur=330
n=0.23
c=0.04MPa
ø=21°
Rf=1.0
ν=0.3
E=14GPa
v=0.25
22.5m

Figure 4.17 Assumptions in two-dimensional analyses of ABV tunnel

hyperbolic parameters cannot be fitted to test results of this soil. For these reasons, it was decided to keep the modulus proposed by Negro and Eisenstein (op.cit.) as a constant value. The thickness of this porous clay deposit was slightly changed in order to better represent the conditions encountered at the instrumented section C. (see Appendix A for details).

The shotcrete lining was simulated as a linear elastic material. The adopted values shown in Figure 4.17 correspond to lower bound values presented by Hoek and Brown (1980:268). These values are in good agreement with the field test results on cylindric samples taken from the ABV tunnel wall presented by Simondi et al. (1982).

The analysis was carried out with the same mesh used for the boundary condition tests, presented in Figure 4.4. Except for some minor differences in the discretization close to the opening, it is the same mesh used by Negro and Eisenstein (op.cit.). This similarity was sought as a way to evaluate the ADINA procedures.

4.4.2 Results of Two-Dimensional Analyses

4.4.2.1 Displacements

Due to the considerable scatter presented by the surface settlement measurements in the field, they were discarded as a criterion for a good fit. Instead, only subsurface settlements are analysed.

The initial linear elastic analysis was executed for a check of ADINA's birth–death procedures. Gravity was applied in the first step, the core elements being 'killed' in the second and last step. The results in Figure 4.18 show a good agreement with those reported by Negro and Eisenstein (op.cit.). This demonstrates that the ADINA procedure produces results similar to conventional 'boundary stress reversal' techniques where the modulus of excavated elements is reduced to a minimum value.

Two runs were carried out with the hyperbolic model. In these runs, the stress release was carried out in 10% increments and lining was installed at %SR=30% and 40% respectively. After lining installation, the 'internal forces' were released and the ground was allowed to close on the lining.

The 'best–fit' results correspond to the 40% stress release and are shown in Figure 4.19. The results correspond closely to the field measurements, suggesting that the set of soil parameters and boundary conditions assumed were appropriate. However, it should be pointed out that the computed deformation occurring after the lining was erected was very small (≃0.8mm) and does not correspond to what was observed in the field (3mm at crown after shotcreting the invert). This could be an indication that the stiffness adopted for the lining was slightly high.

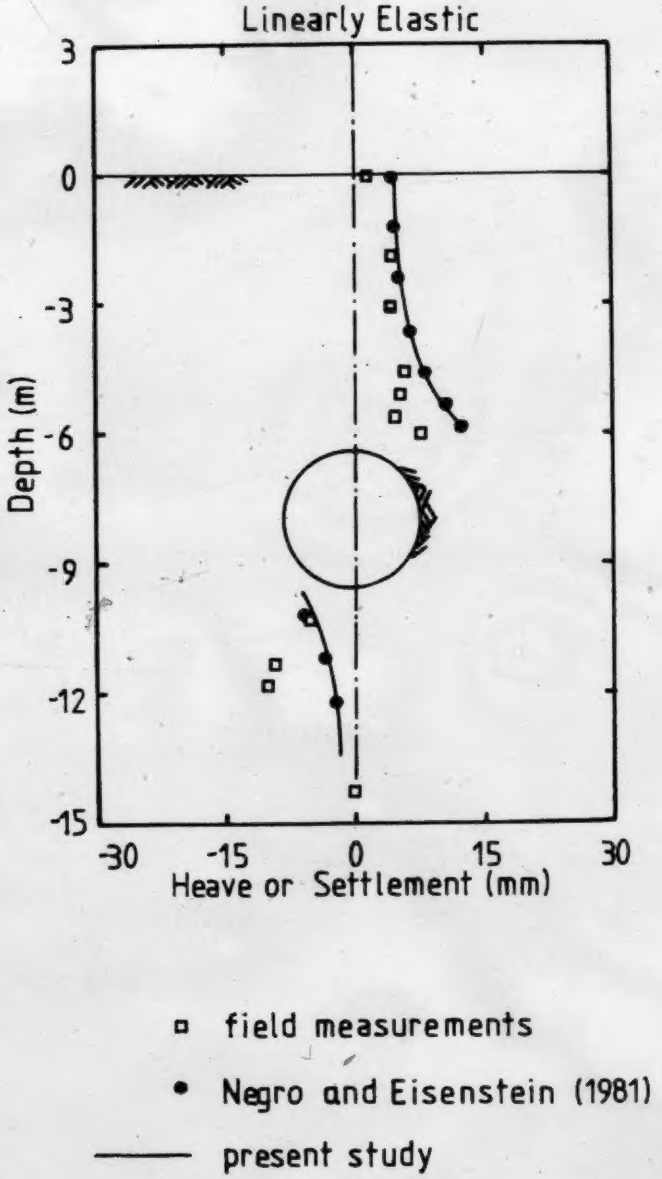


Figure 4.18 Comparison of ADINA results with those published by Negro and Eisenstein (1981)

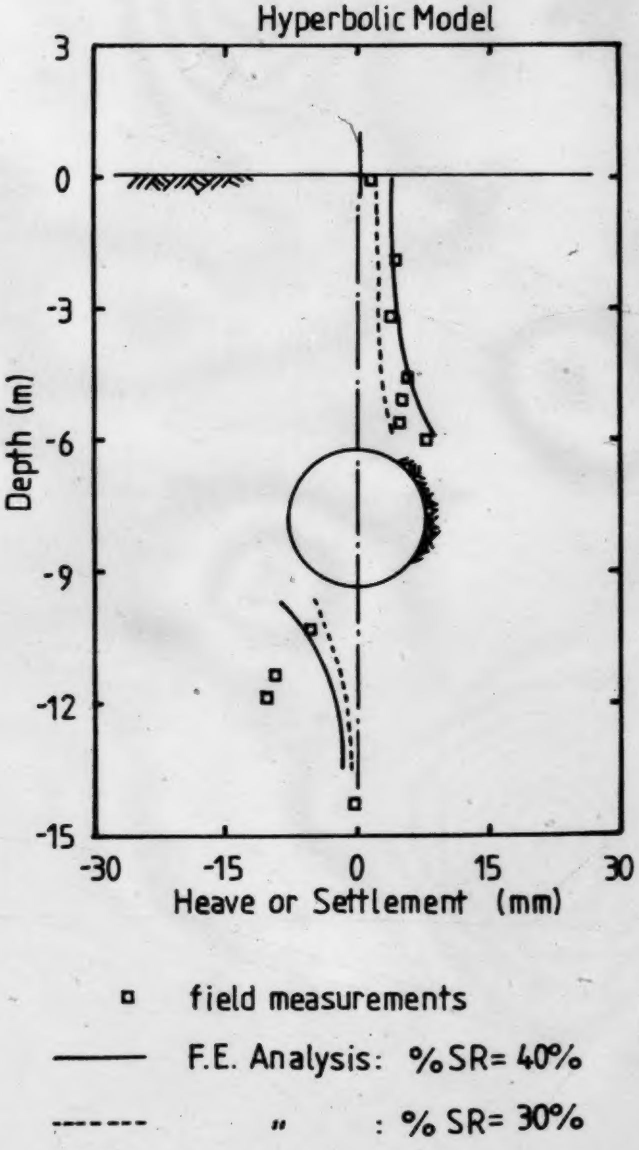


Figure 4.19 Predicted vs. measured displacements: ABV tunnel

4.4.2.2 Comments

The procedure adopted in the 2D linear elastic analysis (i.e., 100% stress release) is straightforward and easily accomplished with ADINA. It is however a very crude approximation of the process of excavation of the tunnel which is better approximated by a gradual stress release. Nevertheless, this procedure may yield good predictions of surface settlements provided convenient soil moduli are adopted and the actual stress-strain behaviour in the field does not depart much from the elastic assumptions.

Regarding the analysis with the hyperbolic model, a good fit of displacements with the field measurements was verified. It should be observed however, that the good 'prediction' was dependent on the assumed percentage of stress release before the lining installation (%SR). A real prediction, carried out before the tunnel is excavated, would require a proper assumption for this parameter.

The %SR used in the analysis is a particularly critical parameter. Increasing its value increases the displacements at the tunnel boundary and hence the surface settlements. In the present work, determination of %SR was carried out by trial and error, turning therefore the two-dimensional finite element analysis into a semi-empirical procedure. Determination of %SR prior to tunnel construction could be achieved through correlations between this parameter and the expected percentage of lost ground V_t (%), since the more stress release is allowed, the larger the crown displacement and consequently V_t, as defined in Chapter 2. Another possibility would be to create a classification system where factors such as soil parameters and excavation method would be related to %SR. In either case, an extensive investigation of several case histories by 2D F.E.M. would be required. This was not attempted in the present work.

It was also found that the procedure of adopting an 'internal support pressure' caused a certain 'disturbance' in the elements surrounding the opening. This is because the applied nodal loads, which were calculated manually, do not correspond exactly to the loads created by the birth-death option. Hence the equilibrium at the tunnel boundary could not be fully satisfied. For this reasons, the displacements at the tunnel boundary were discarded.

Two ways to avoid the problems reported above are to:

1. Refine the finite element discretization around the tunnel. This would decrease the stress differences at the element boundaries.
2. Establish an 'in-core' pre-processor in ADINA which would calculate the exact nodal loads from the condition of equilibrium of the elements and

gradually release these loads in small steps.

Neither of these techniques was attempted because the agreement between calculated and observed values was considered satisfactory. Furthermore, modification of the ADINA code was not considered feasible due to the restricted time frame of this study. Such expedients may however be necessary for future research involving the technique reported above if more precise values of the stresses and displacements at the boundary are required.

4.5 Analysis of Large Cross-Section Tunnels

The objective of this section is to analyse, by using the FEM, the relative amount of surface settlements provoked by two different staged excavation schemes presented in Chapter 3. It is recognized that the excavation of a large cross-section tunnel is a truly three-dimensional process. However, three-dimensional analyses of such tunnels are extremely complex and would require a considerable time effort.

A simple two-dimensional analysis, regardless its clear limitations, allows an estimate of the relative performance of different construction schemes in terms of ground movements. However, it does not permit a proper assessment of problems like face stability or development of lining pressures.

4.5.1 Statement of the Problem

The analyses are aimed at comparing settlements due to excavation of a hypothetic tunnel 12m wide and 9m high with about $80m^2$ of cross-sectional area. The soil cover above the tunnel crown is taken as 10m.

A comparison is intended for two construction schemes: type T1 ('heading and bench') and type T4 ('side galleries') as reviewed in Chapter 3. The assumed geological profile is depicted in Figure 4.20. The adopted geotechnical properties are also shown and correspond roughly to extreme values reported for the Edmonton till by Eisenstein (1981). Two sets of analyses are carried out for these different types of soil which are arbitrarily termed 'hard' and 'soft' till. The upper horizon properties correspond to average values for Lake Edmonton clay reported by Eisenstein (op.cit.).

4.5.2 Finite Element Simulation

The stress-strain behaviour of the soil was represented by the von Mises model described in the preceeding Sections. The value of the yield stress σ_y was chosen as the value of the uniaxial compressive strength of the soils $(\sigma_y=q_u=2c_u)$ and the hardening modulus was arbitrarily taken as E/10, where E is the modulus in the elastic portion of the stress-strain curve. Successful use of this approach has been reported by Dysli et al. (1979) and Dysli and Fontana (1982).

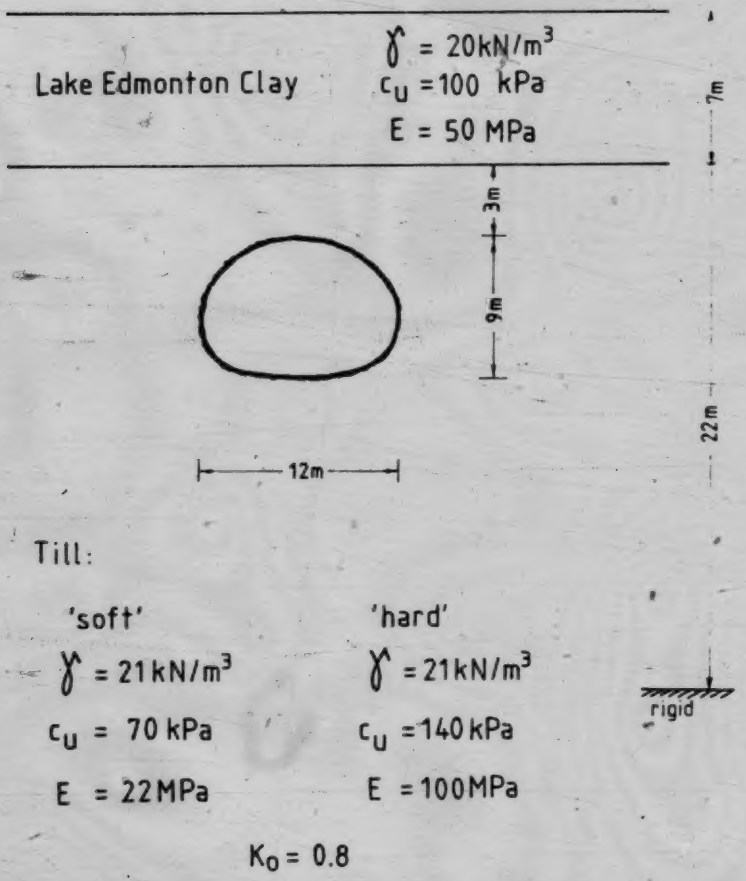

Lake Edmonton Clay
γ = 20kN/m^3
c_u = 100 kPa
E = 50 MPa
7m
3m
9m
12m
22m
Till:
'soft'
γ = 21kN/m^3
c_u = 70 kPa
E = 22MPa
'hard'
γ = 21kN/m^3
c_u = 140kPa
E = 100MPa
rigid
K_0 = 0.8

Figure 4.20 Hypotheses for analysis of large cross-section tunnels

The shotcrete lining was represented by beam elements and assumed to behave linearly elastic with E=10GPa and ν=0.25. The thickness of the inner wall of the side galleries was assumed to be 15cm, while the thickness of the external wall was taken as 25cm. The finite element mesh used is shown in Figure 4.21 (dimensions were selected according to the parametric study reported in Section 4.2.4). Due to the comparative nature of the study, it was decided that a coarse mesh would suffice. Tunnel construction simulation is shown in Figures 4.22a and 4.22b. It should be noted that due to impossibility of activating and de-activating an element in the same run, the inner walls of the side galleries in scheme T4 cannot be removed. Step 8 in Figure 4.22b is therefore a fictitious step, whose effect was estimated as follows:

1. Run problem with linear elastic soil properties and fully excavate opening with internal walls.
2. Run same problem without internal walls.
3. The difference in displacements in both cases is added to the real problem.

4.5.3 Results of Comparative Analyses

For each construction step, the corresponding nodal displacements were obtained. Figures 4.23 and 4.24 show the maximum settlement profiles for the soft and hard tills respectively. Also shown are the settlements occurring during heading excavation (step 1 in the T1 scheme and step

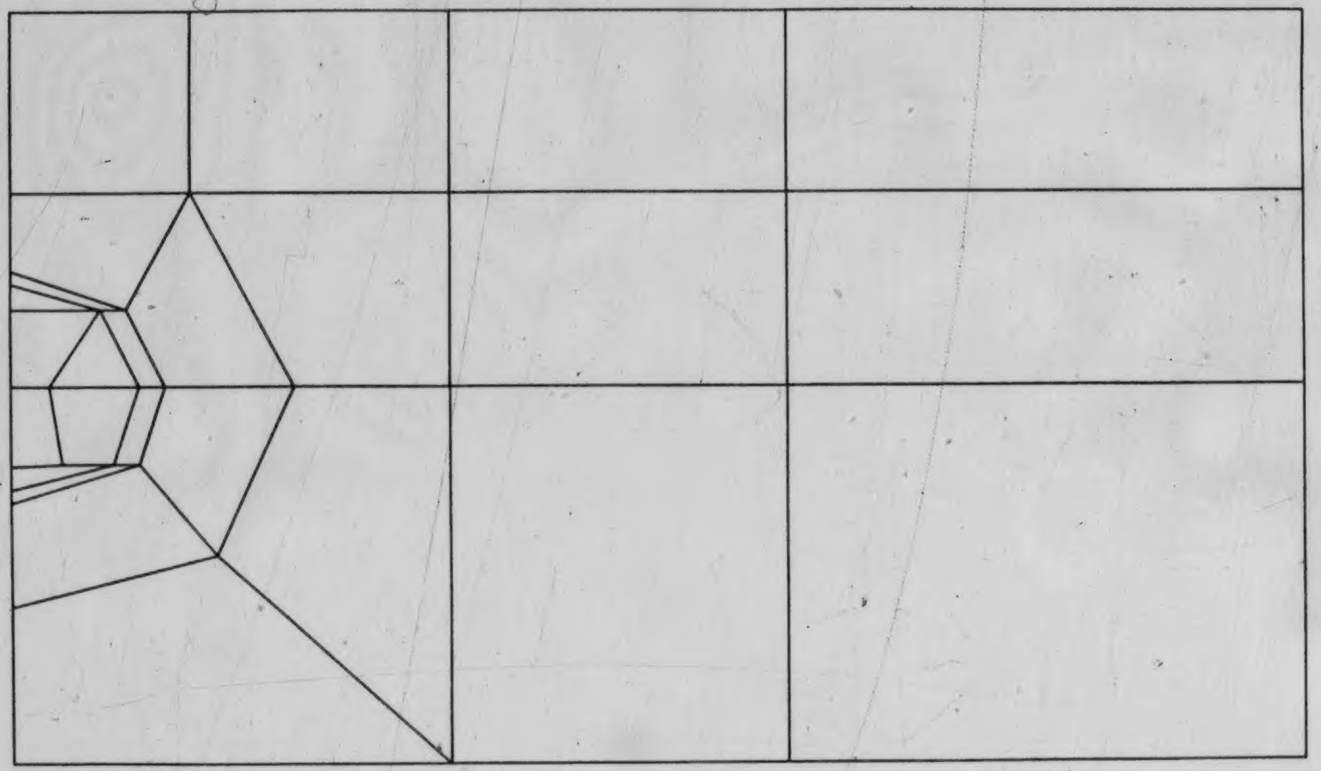
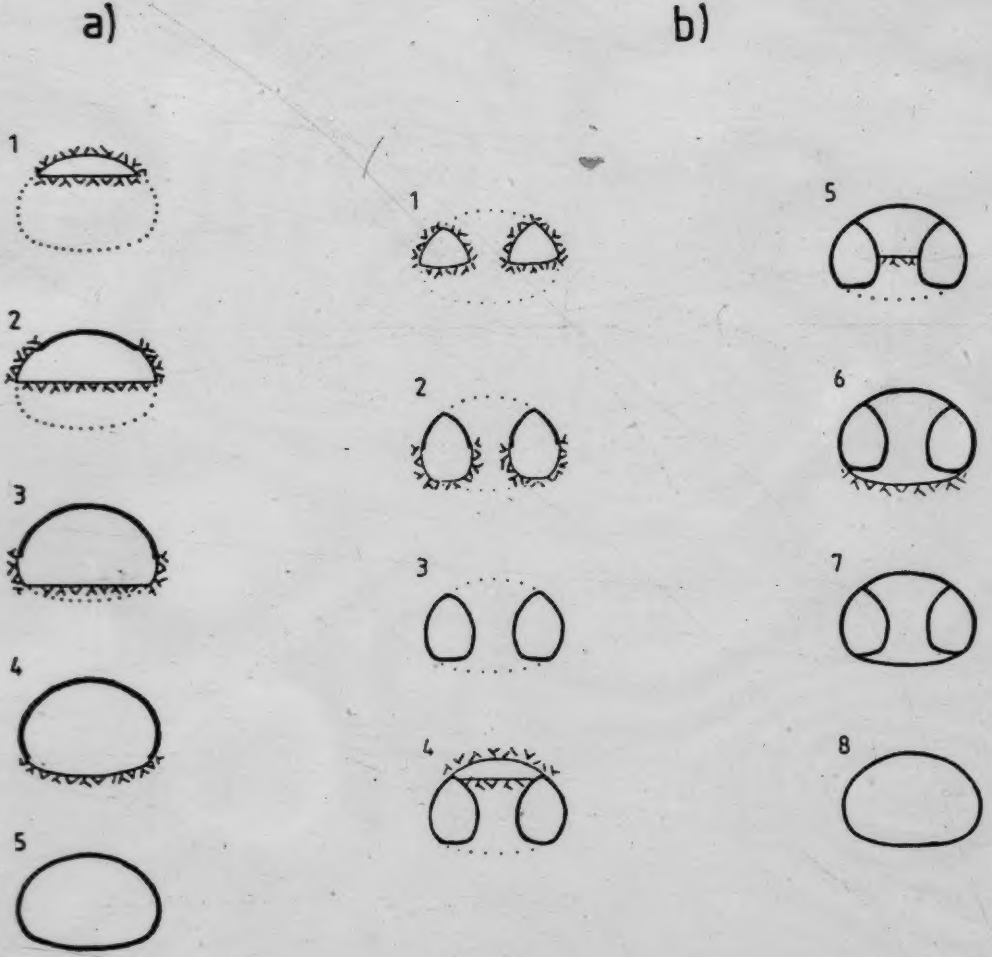


Figure 4.21 Finite element mesh for analysis of large cross-section tunnel


a)
b)
1
2
3
4
5
1
2
3
4
5
6
7
8

T1

T4

Figure 4.22 Construction steps in finite element simulation

3 in the T4 scheme).

It should be pointed out that by virtue of the simplified two–dimensional representation, the ground mass above the tunnel experiences some heave during invert excavation and immediate lining erection. Therefore, the maximum settlements occurred at a stage prior to the final step, what is unlikely to happen in a proper three–dimensional analysis. This is illustrated in Figure 4.25 which shows a schematic evolution of vertical displacements of a point at the tunnel crown for both construction schemes. The distances between construction steps have been arbitrarily selected and are shown in the lower part of Figure 4.25. The displacements ahead of the face have been assumed equal to one third of the elastic displacement given by 'hole-in-a-plate' solutions. The onset of displacements has been assumed to occur at 1.0 diameter ahead of the face.

4.5.4 Interpretation

The striking conclusion that can be derived from inspection of Figures 4.23 and 4.24 is that regardless of soil properties, the scheme T4 has a better performance in terms of maximum surface settlements or maximum distortion. In terms of absolute values however, the differences in the case of the stiffer soil are less significant. This suggests that in this case the criteria for selection of either one of these schemes could be linked to factors other than the

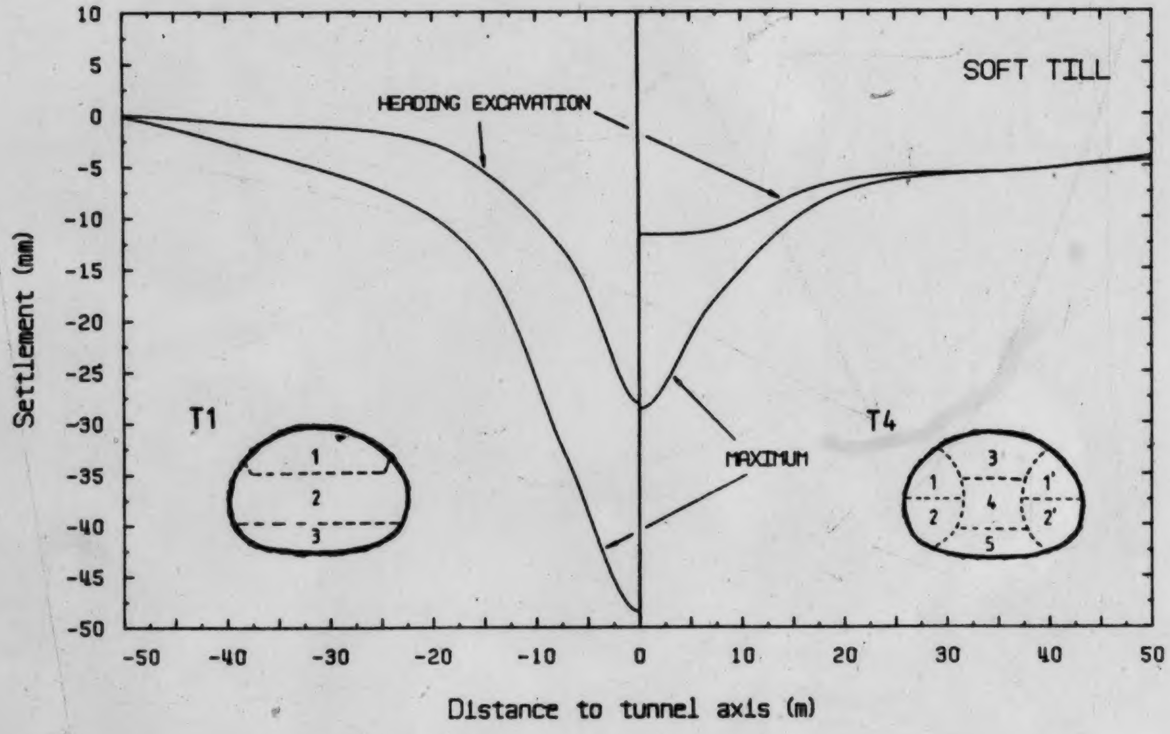


Figure 4.23 Predicted surface settlements: soft till

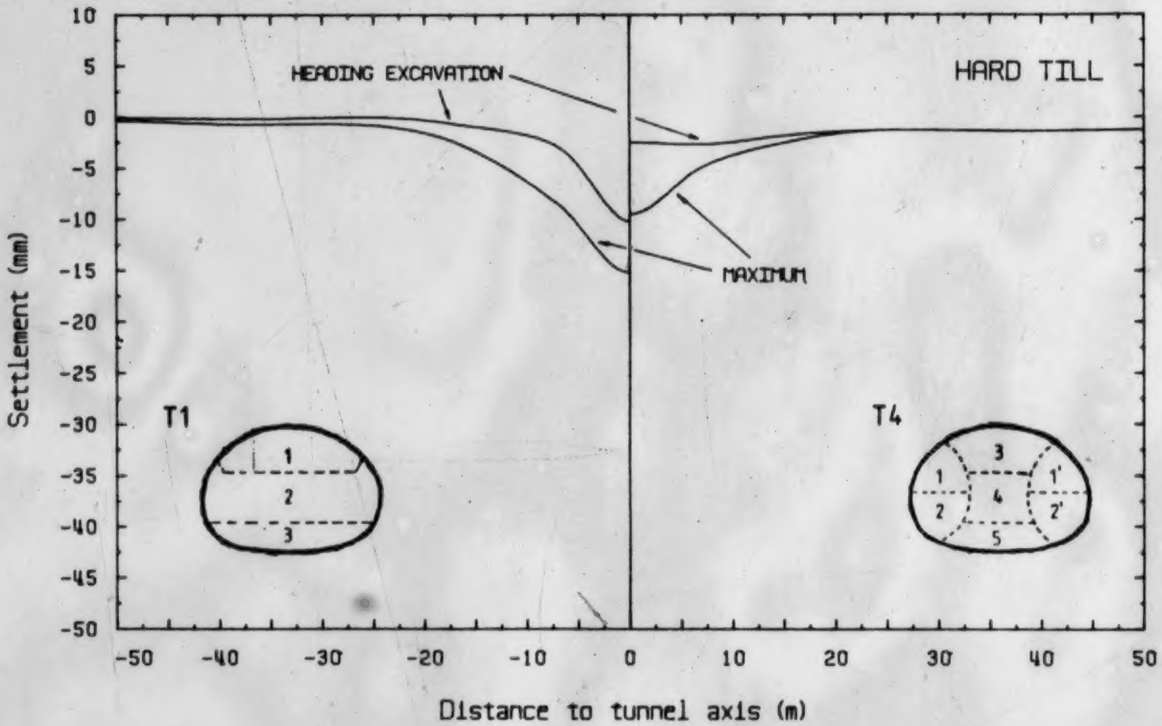


Figure 4.24 Predicted surface settlements: hard till

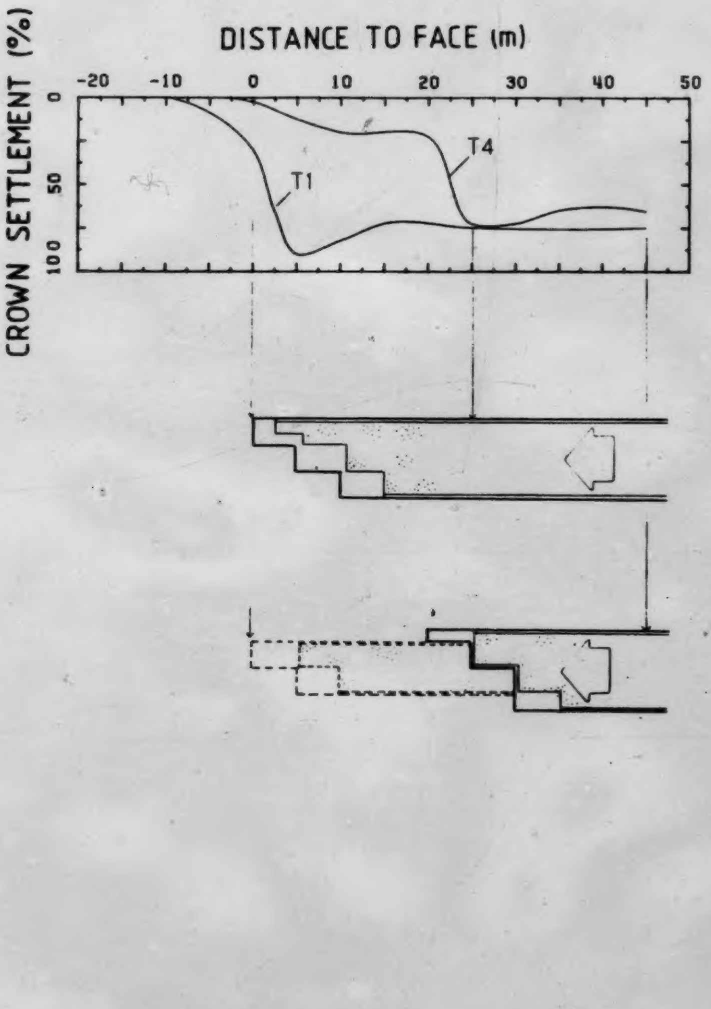


Figure 4.25 Longitudinal displacement evolution for a point at the tunnel crown derived from 2D analyses

performance in terms of surface displacements, such as those reported in Section 3.3, Chapter 3.

Another feature of interest is the amount of settlement caused by excavation of the heading portion (step 1 in the T1 scheme and step 3 in the T4 scheme). In both cases and specially in the case of the softer soil, the percentage of the maximum settlement generated by this excavation step is very significant. In practical terms, this shows that excavation of the heading in the field should be carried out with special care, perhaps with the adoption of a central core, as illustrated in Figure 4.26[23].

4.6 Conclusions

For the sake of clarity, it is appropriate to separate the conclusions of this chapter into different sections. Although the main objective of the study was to evaluate the application of two-dimensional finite element techniques to the NATM, considerable attention was paid to modelling criteria and to features of the program ADINA.

4.6.1 Modelling Criteria

A review of recent literature on finite element analysis of shallow tunnels has shown that details on the techniques used to simulate excavation are often obscured by a formal presentation. Due to the lack of published modelling criteria for shallow tunnels, several runs were

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[23] This is an expedient frequently used in NATM works (e.g. Nixdorf, 1980:43).

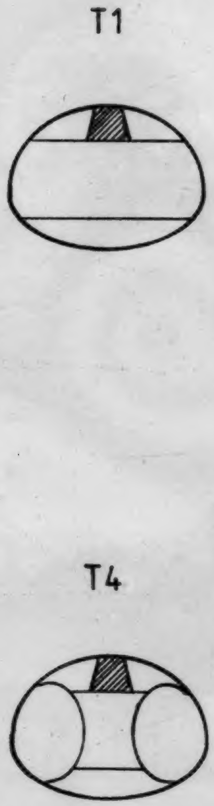

Figure 4.26 Use of central core to minimize displacements due to excavation of the heading

executed in order to assess the influence of mesh size and boundary conditions in the analyses presented herein. Important points which emerged from these parametric studies are:

1. The distance to the lateral boundary of the mesh did not influence the settlement profile provided it was placed at more than 8 diameters from the tunnel centerline (rollers were used to simulate the boundary conditions at both vertical boundaries).
2. The choice of the boundary condition (i.e., rollers or fixed boundaries) at the bottom rigid boundary did not affect the predicted settlements.
3. Analyses which assume the E modulus constant with depth cannot provide unique predictions of settlements due to tunnel excavation. These displacements will be controlled by the depth of the bottom rigid boundary.
4. The position of the bottom rigid boundary becomes much less critical when an increase in the E modulus with depth is assumed. This enhances the necessity of realistic assumptions as to how the stiffness actually varies with depth in analysis of shallow tunnels.

4.6.2 ADINA

Special attention was given to some features of ADINA. Since two-dimensional modelling of the advance of a shallow tunnel is not straightforward, some specific techniques (which used the available ADINA features) were developed at the expense of considerable time and effort. It is hoped that these initial steps can provide easier ways for future users. Some important points are:

1. The 'birth-death' option alone does not allow appropriate modelling of an advancing tunnel in two dimensions due to the fact that the stress release at the tunnel excavation boundary cannot be made gradually.
2. Using ADINA's time functions, an alternate procedure was developed to simulate tunnel advance (Section 4.3.3). This procedure yielded satisfactory results for the present study, where matching the settlement profile was achieved after a 40% stress release. However, at higher percentages of stress release it was difficult to maintain the equilibrium at the tunnel wall. Alternatives for future improvements were suggested in Section 4.4.2.2.

4.6.3 Analysis of ABV tunnel

A considerable improvement with respect to the previous study by Negro and Eisenstein (1981) was obtained with the application of the lining after a certain amount of boundary stress release, as defined in Section 4.3.3). The use of a hyperbolic soil model which accounted for the variation of stiffness with depth is also believed to have contributed to this improvement.

An important remark refers to the 'percentage of stress release' (represented by %SR in Figures 4.16 and 4.19) allowed before lining installation. The predicted settlements were dependent on this parameter which was not known beforehand and was determined by trial and error. Appropriate 'calibration' will be clearly required before two-dimensional finite element analyses using this technique can provide unique predictions. It was suggested that the calibration factor can be achieved through correlations between the %SR and the percentage of lost ground V_t as defined in Chapter 2. Another possibility suggested was the creation of a classification system in which %SR would be one of the parameters. In either of these cases, the 2D finite element analysis of advancing tunnels would become a semi-empirical procedure of predicting settlements.

4.6.4 Large Cross-Section Tunnels

Two-dimensional finite element analyses were carried out to compare relative performance of two staged excavation construction schemes. Two extreme soil conditions believed to represent the Edmonton till were analysed. Three important conclusions resulted from these analyses:

1. The staged excavation scheme with side galleries (T4) is superior to the heading and bench scheme (T1) in terms of maximum surface settlements generated. The slope of the settlement trough for scheme T4 was also found to be flatter, confirming field experiments reported by Krischke and Weber (1981).
2. In the stiffer soil, termed 'hard till', the differences

in settlements between the two schemes were less significant. This suggests that selection criteria other than the geotechnical performance in terms of settlements could control when excavating large cross-section tunnels in such soils.

3. Excavation of the heading portion is responsible for a significant amount of the total settlement. When surface settlements cannot be tolerate, it might be advisable to adopt a central core, as shown in Figure 4.26, 'staging' the excavation at the heading in both T1 and T4 schemes.

5. THREE-DIMENSIONAL FINITE ELEMENT ANALYSES

5.1 Introduction

This chapter deals with applications of the three-dimensional finite element method to analysis of shallow tunnels excavated using the NATM. Initially the method is used to model a case history, with the numerical results being compared to actual field measurements.

A further contribution deals with the three-dimensional stress-displacement behaviour near the face of an advancing shallow tunnel. Two three-dimensional runs are executed with the lining installed at different distances from the face. The results are studied within the framework of the convergence-confinement approach, where soil and lining behaviour are represented by characteristic curves defined in Chapter 1. All analyses were carried out with the program ADINA.

5.1.1 General

Most finite element analyses of shallow tunnels published in the literature to the present date are two-dimensional representations. While in several problems in geotechnical engineering (e.g. 'long' dams and excavations) this approximation is fairly reasonable, such is not the case of tunnels excavated by the NATM, where most of the 'action' takes place near the face. This is illustrated in Figure 5.1, which depicts two and

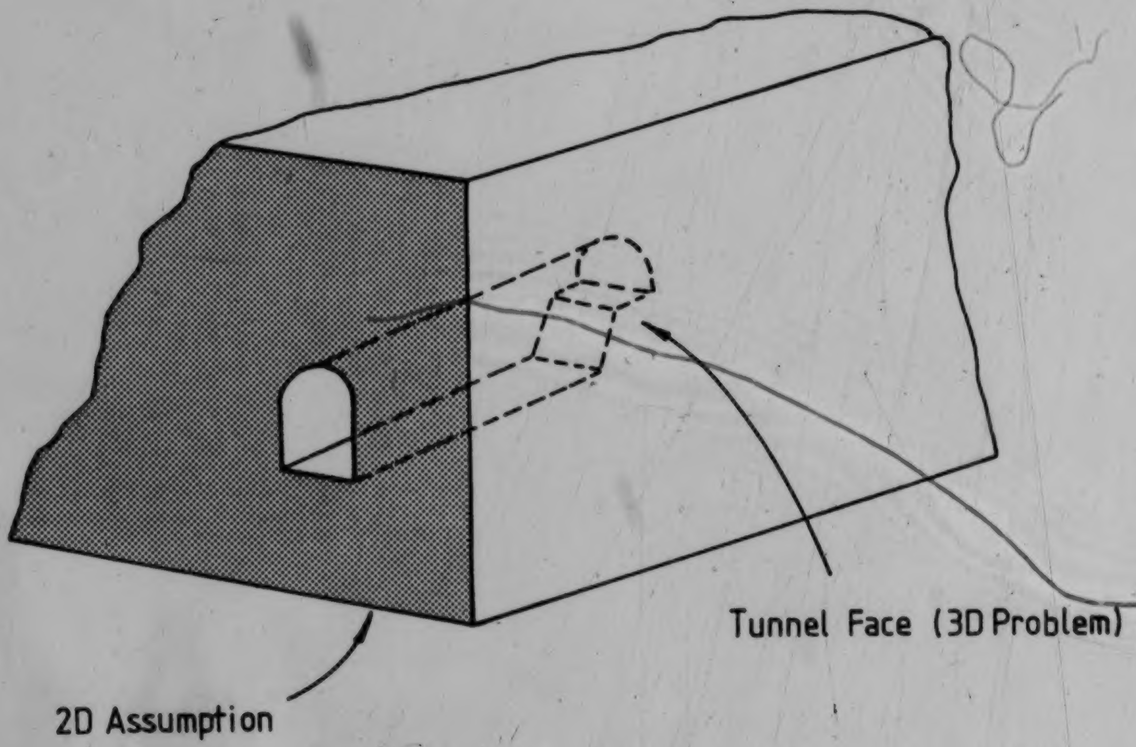
Tunnel Face (3D Problem)
2D Assumption

Figure 5.1 Two and three-dimensional representations of an advancing tunnel

three-dimensional representations of the same tunnel.

5.1.2 Modelling Criteria

This section complements the ideas approached in Section 4.2, Chapter 4 with respect to the 3D analysis.

5.1.2.1 General

There is great demand for computer storage capacity and on time for input preparation and output interpretation associated with three-dimensional analyses. Therefore, studies to evaluate the factors influencing the analyses such as those presented for the two-dimensional case (Section 4.2, Chapter 4) could not be carried out. Selection of mesh size and appropriate simulation techniques were based on literature review and are briefly outlined in the following sections.

5.1.2.2 Mesh Discretization

Due to the limited number of properly documented three-dimensional finite element studies available at the onset of the present study, it was decided to adopt dimensions close to those specified in the 'German Recommendations for Underground Constructions in Rock' (DGEG, 1979:198). The mesh subdivision was chosen as a compromise between relatively coarse meshes (e.g. Descoeudres, 1974) and more refined studies (e.g. Gartung et al., 1979).

The three-dimensional continuum is discretized according to the following principles:

1. The grid is subdivided into a number of 'slices' and is kept as simple as possible for easier visual interpretation. This is extremely important when no pre- or post-processors are available, as in the present study.
2. A 'test section' close to the central part of the mesh is selected for analysis, similar to an instrumented section in the field. Stresses and displacements are analysed for this section only and the normally lengthy 3D computer output is eliminated[24].
3. For the initial excavation steps which take place far from the 'test section' a coarse mesh subdivision is used. For steps closer to the test section the number of slices is increased, to better simulate the sequence of excavation and support erection of the actual tunnelling process.

Figure 5.2 illustrates the three-dimensional discretization in a schematic manner. Eight node elements were used to represent the ground and the lining in all analyses presented in this chapter. They are the simplest form of three-dimensional elements available in ADINA and were chosen to reduce costs and simplify the nodal numbering sequence, allowing easier manipulation of input and output information.

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[24] This test section was implemented only in the analysis of the case history presented in Section 5.2.

5.1.2.3 Construction Simulation

Excavation and lining erection were carried out by the 'birth–death' procedure outlined in Chapter 4. Although this ADINA option would allow a closer simulation of the actual NATM excavation sequence, the number of necessary time steps would be high, resulting in higher costs. Therefore, the simplified excavation sequences illustrated in Figure 5.3 were adopted. It should be noted that each three-dimensional excavation step represents five actual steps and that the lining is erected simultaneously with excavation.

The observations about the necessity of a 'thin layer' of elements at the excavation boundary and to the reactivation of lining elements explained in Section 4.2.2, Chapter 4, apply here as well. No tests were carried out to check the influence of the thin layer boundary on the results but the conclusions derived from the two–dimensional verifications can be clearly extended to the three–dimensional case.

5.1.2.4 Material Model

All three–dimensional analyses were carried out assuming linearly elastic isotropic behaviour for soil and lining. The use of an elastic model is a recognized oversimplification of soil behaviour. However, in the investigations presented in this thesis more attention

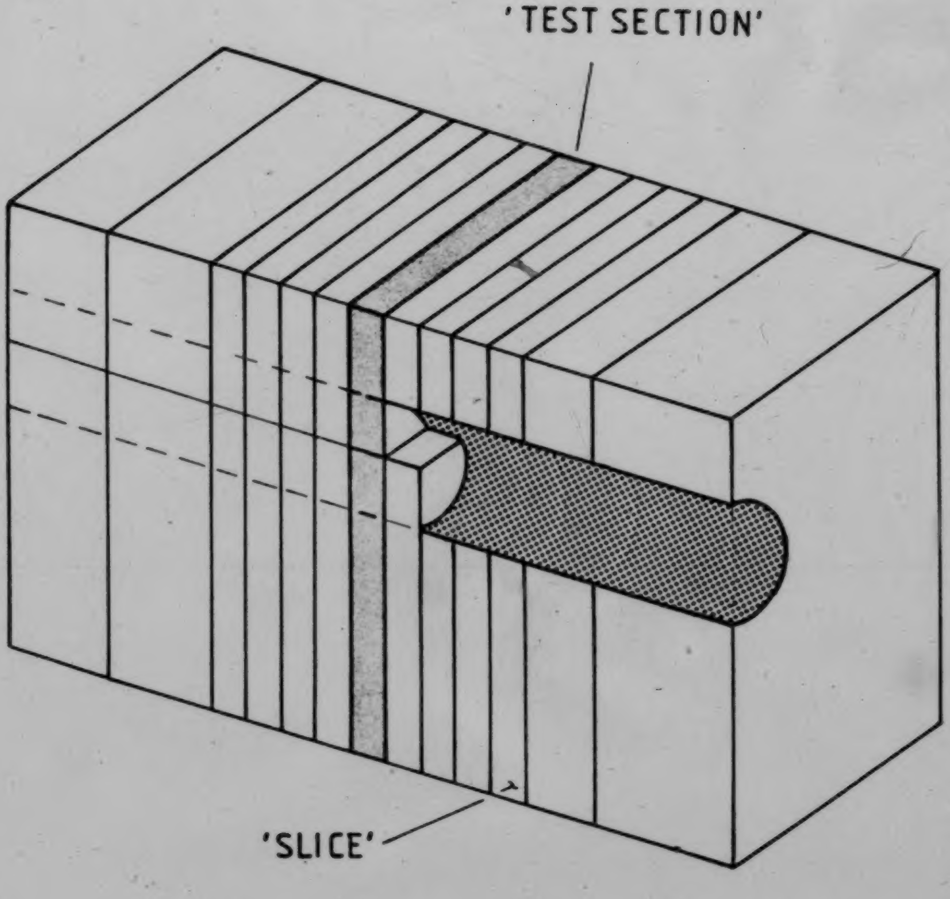
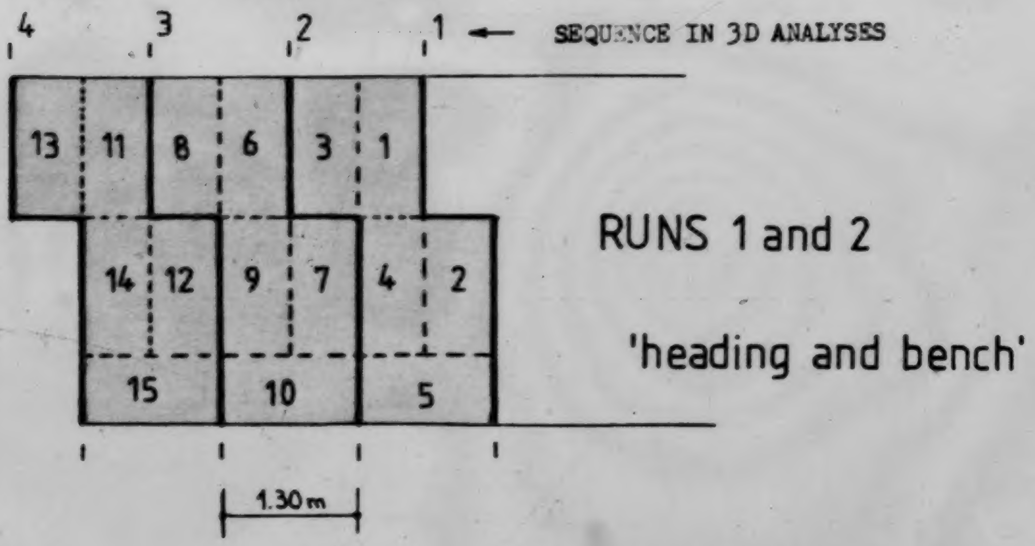
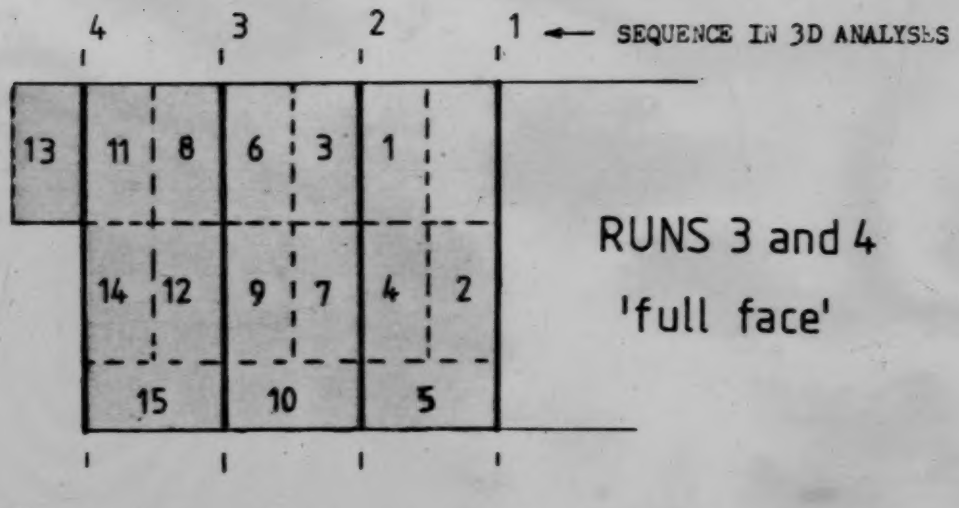

'TEST SECTION'
'SLICE'

Figure 5.2 Schematic subdivision of three-dimensional continuum


4
3
2
1 ← SEQUENCE IN 3D ANALYSES
13
11
8
6
3
1
14
12
9
7
4
2
15
10
5
RUNS 1 and 2
'heading and bench'
1.30 m


4
3
2
1 ← SEQUENCE IN 3D ANALYSES
13
11
8
6
3
1
14
12
9
7
4
2
15
10
5
RUNS 3 and 4
'full face'

NOTE: Numbers within shaded areas represent actual sequence of excavation

Figure 5.3 Actual NATM construction sequence vs. finite element simulation

was paid to particular aspects of the 3D behaviour during tunnel excavation than to the constitutive law. The high costs involved in non-linear analyses also contributed to justify this assumption[25]. Moreover, in the ABV tunnel analysed in Section 5.2, the ground losses were very small, notably those ahead of the face. Therefore, a great departure from the elastic behaviour was not expected. Indeed, as will be seen in the following sections, the displacements ahead of the face of this particular tunnel were matched even with the assumption of linear elasticity.

5.2 Analysis of São Paulo ABV Tunnel

This is a case history described in Appendix A and analysed in two-dimensions in Chapter 4. The two-dimensional analyses yielded satisfactory predictions of cross-sectional displacements, the three-dimensional analysis being used to study the longitudinal development of stresses and displacements with face advance.

Two three-dimensional runs were carried out. The initial run (termed Run 1) was a 'pilot run' and evaluated the degree of difficulty involved in the analysis. The analysis also examined the adequacy of the selected simulation technique. The parameters for Run 2 were then selected, based on the results of the initial run. The results of this second run were the best achieved in this study.

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[25] Simple three-dimensional tests indicated that the cost of analyses with non-linear models could be more than ten times higher than the linearly elastic analyses.

5.2.1 Material Properties and Geometry

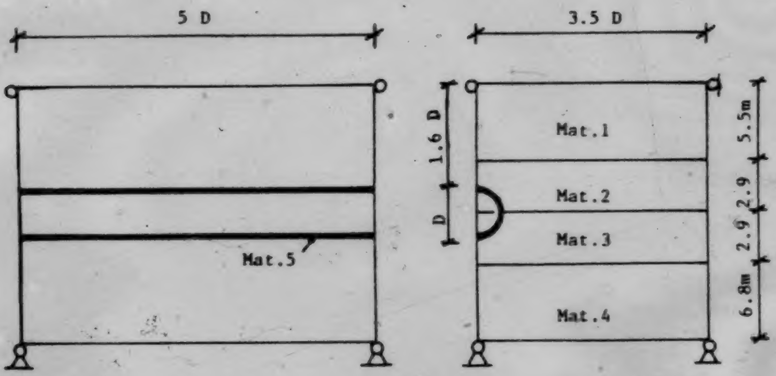
Material properties and geometry are illustrated in Figure 5.4. In Run 1, the 'pseudo-modulus' for the variegated soil determined by Negro and Eisenstein (1981) was adopted for the whole mesh, with no variation assumed with depth. Poisson's ratio was assumed to be 0.3, yielding a K_0 value of 0.43. In Run 2, the moduli adopted were the initial tangent moduli (E_i) for the variegated soil given by Equation 2 in Figure 4.12, Chapter 4. The value of σ_3 was taken at the centre of each layer. Hyperbolic parameters for this soil are outlined in Appendix A. The K_0 value in Run 2 was increased, based on a comparison of the results of Run 1 with field measurements, as explained in the following sections.

5.2.2 Mesh Discretization and Excavation Simulation

The mesh used is illustrated in Figure 5.5, a typical 'slice' of the mesh being shown in Figure 5.6. It should be noted that in Run 2 the mesh was slightly modified in order to accomodate horizontal layers. The excavation sequence is as follows:

Step 1: Establishment of initial conditions (i.e, application of gravity)

Step 2: Full face excavation of slice 1


5 D
3.5 D
Mat.5
1.6 D
D
Mat.1
Mat.2
Mat.3
Mat.4
5.5m
2.9
2.9
6.8m

RUN 1

Material	E (MPa)	ν
1	61.	0.3
2		
3		
4		
5	14000.	0.25

Ko=0.43

RUN 2

Material	E (MPa)	ν
1	12.	0.43
2	31.5	
3	34.5	
4	37.2	
5	14000.	0.25

Ko=0.75

Figure 5.4 Geometry and material properties used in back analyses of ABV case history

Step 3: Full face excavation of slice 2; shotcrete at slice 1

Step 4: Heading excavation of slices 3, 4 and 5; shotcrete at slice 2

Step 5: Heading excavation of slices 6 and 7; bench excavation of slices 3, 4 and 5; shotcrete at slices 3, 4 and 5 (heading) and slice 3 (bench)

Step 6: Heading excavation of slices 8 and 9; bench excavation of slices 6 and 7; shotcrete at slices 6 and 7 (heading) and slices 4 and 5 (bench) – FACE AT TEST SECTION

Step 7: Heading excavation of slices 10 and 11; bench excavation of slices 8 and 9; shotcrete at slices 8 and 9 (heading) and slices 6 and 7 (bench)

Step 8: Heading excavation of slices 12 and 13; bench excavation of slices 10, 11 and 12; shotcrete at slices 10 and 11 (heading) and 8 and 9 (bench)

Step 9: Full face excavation of slice 14; bench excavation of slice 13; shotcrete at slices 12 and 13 (heading) and slices 10, 11 and 12 (bench)

Step 10: Full face excavation of slice 15; shotcrete at slice 14 and slice 13 (bench)

5.2.3 Comparison with Field Measurements

As explained in Appendix A, the measurements of displacements around the ABV tunnel were restricted to

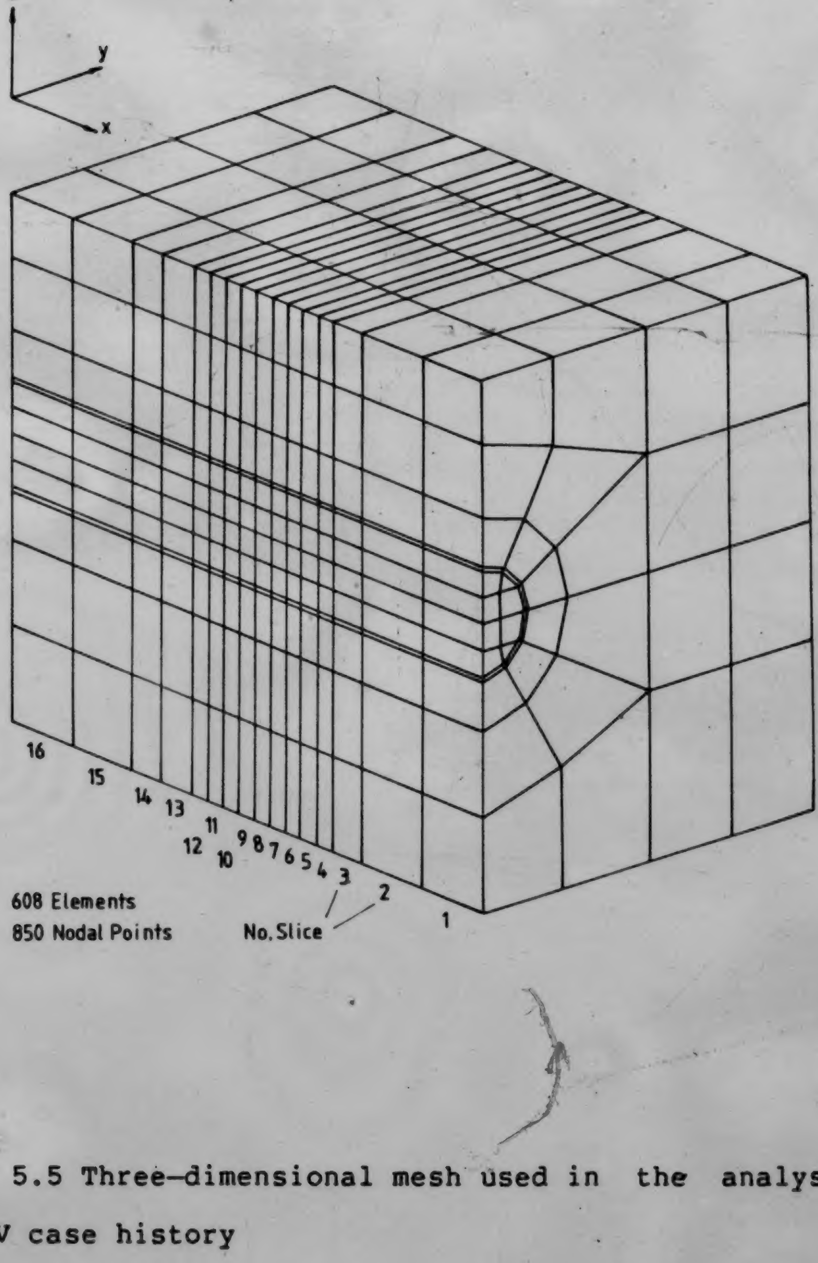

z
y
x
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
608 Elements
850 Nodal Points
No. Slice

Figure 5.5 Three-dimensional mesh used in the analysis of the ABV case history

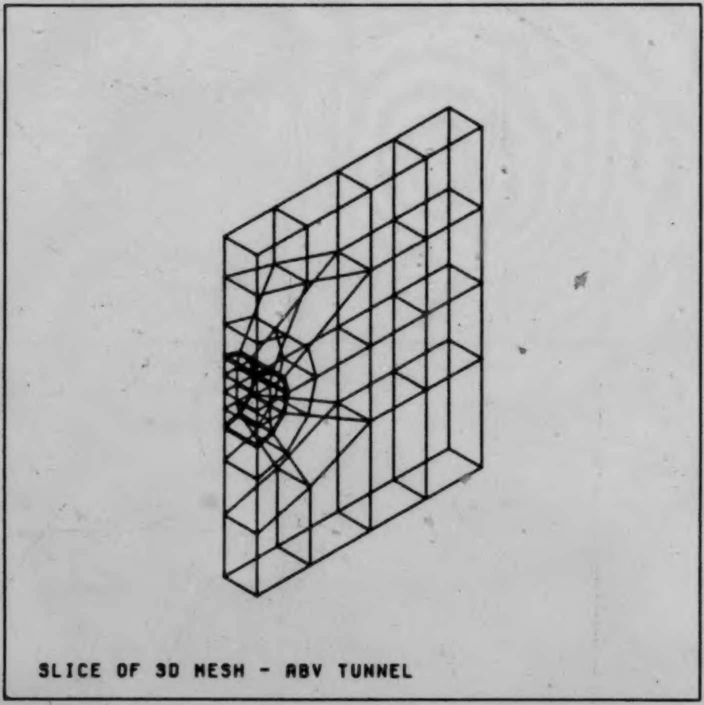

SLICE OF 3D MESH - ABV TUNNEL

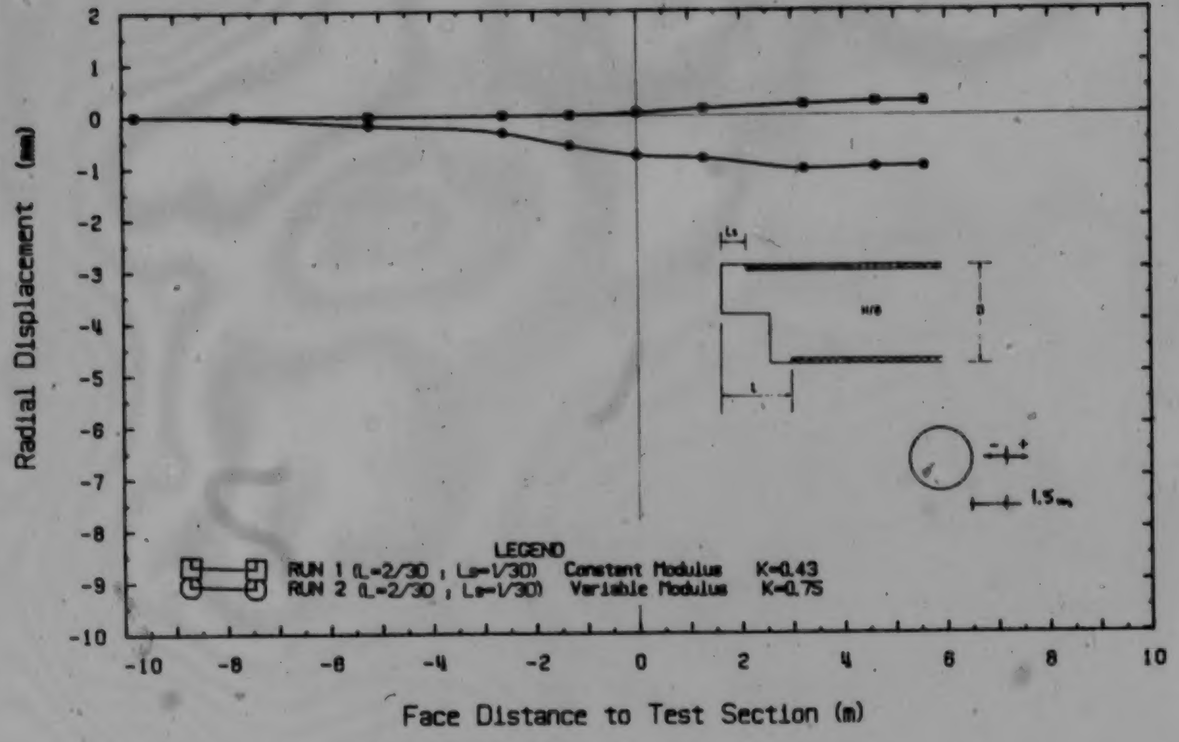
Figure 5.6 Typical slice of 3D mesh – ABV study

surface settlements, roof levelling, convergence measurements and subsurface movements measured by extensometers. Comparison between the field measurements and results of the finite element analyses allows an evaluation of the adequacy of the parameters selected.

It should be pointed out however that the comparisons presented in the following sections were made for points located at about 1.0m (crown and invert) to 1.5m (springline) away from the mesh opening. This was necessary because the displacements and stresses at tunnel boundary were affected by numerical innacuracies, as commented at the end of this chapter.

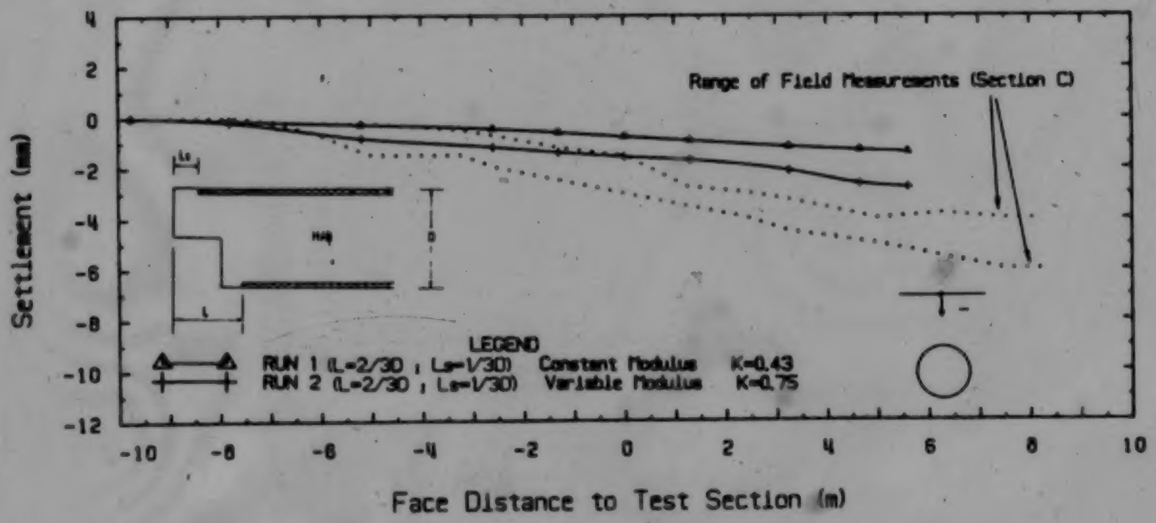
5.2.3.1 Horizontal Displacements

The evolution of horizontal displacements with the face advance is shown in Figure 5.7. In an actual situation, this would correspond to readings of a slope indicator installed close to the springline level. Results of Run 1 show that there is practically no horizontal movement before the face passes the test section. After the face passes by, there are slight outward horizontal movements. These were attributed to the K_0 value chosen for Run 1 and do not correspond to what was observed in the field. Although no slope indicator was used, results of convergence measurements published by Negro and Eisenstein (1981) showed that this tunnel displayed an inward movement at the springline.



SPRINGLINE DISPLACEMENT - Runs 1 and 2

Figure 5.7 Radial displacements at springline - ABV tunnel



SURFACE SETTLEMENTS - Runs 1 and 2

Figure 5.8 Surface settlements – ABV tunnel

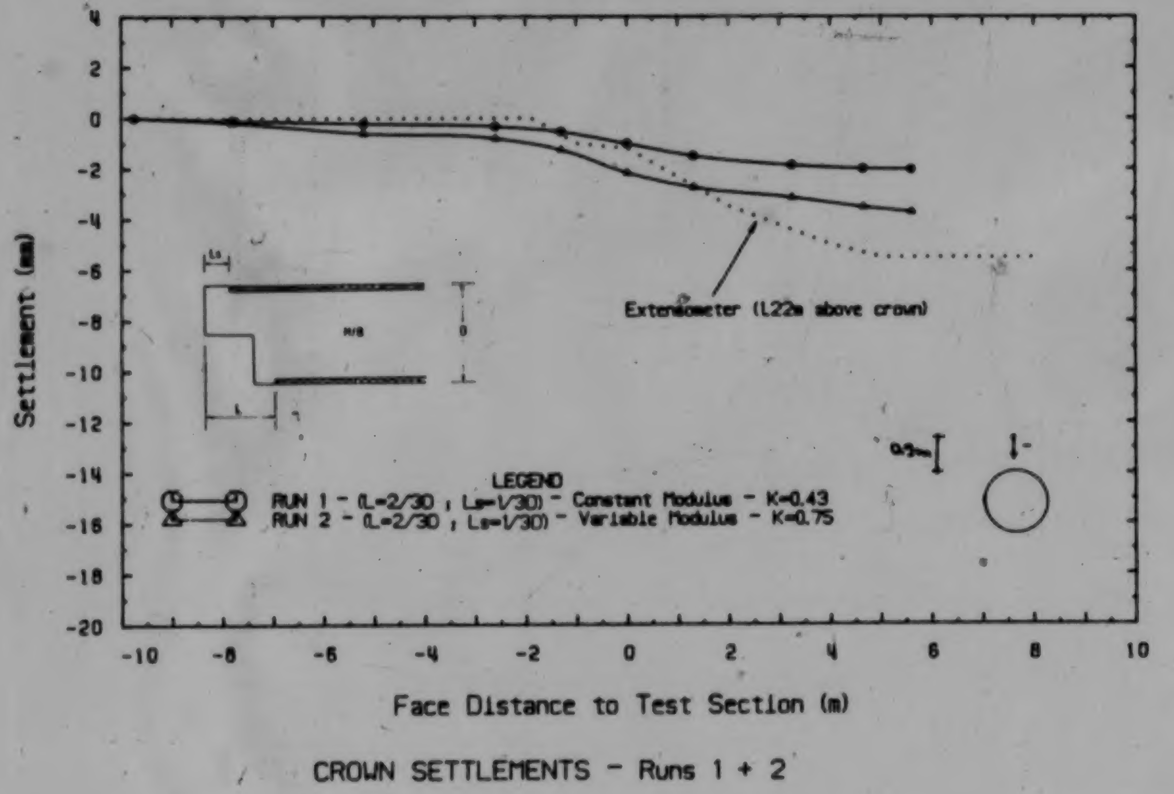
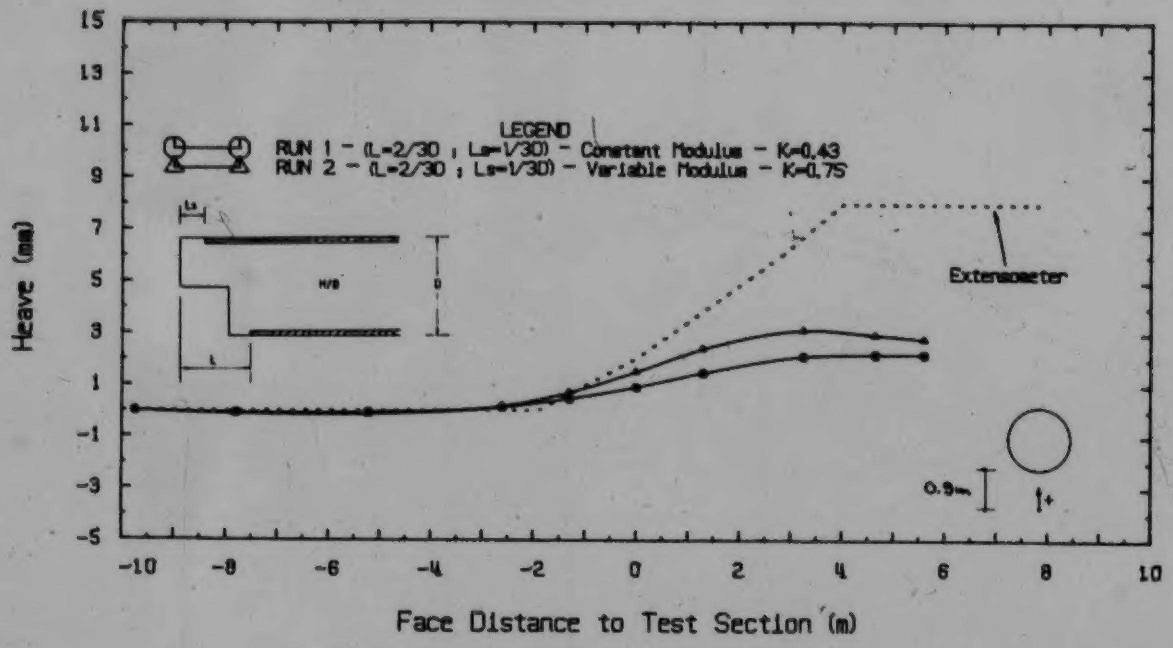


Figure 5.9 Crown settlements – ABV tunnel



INVERT HEAVE - Runs 1 + 2

Figure 5.10 Invert heave - ABV tunnel

For Run 2, the K_o value was increased to 0.75, based on considerations regarding the ABV field case presented in Appendix A . It was observed that by the time the face reached the test section, a radial inward displacement had already occurred[24]. This displacement is slightly hindered by the placement of the lining at the top heading and tends to stabilize after the invert is closed.

5.2.3.2 Vertical Displacements

Surface and subsurface vertical displacements were measured in the field using ordinary level surveys and multipoint extensometers. The results of these measurements are partially presented in Appendix A and show settlement for points at the surface and heave at the invert level.

The plots of vertical displacements are presented in Figures 5.8, 5.9 and 5.10. It should be noted that the displacements ahead of the face can be reasonably matched by the linear elastic model. However, after the face passes, the displacements measured around the tunnel are much larger than those calculated. This indicates that the behaviour ahead of the face does not depart much from the linear elastic assumption. As the face passes by, an increase in the mobilization of ground shear strength will result in larger displacements, which cannot be matched because the elastic moduli are not updated for these excavation steps. Matching the field measurements, in this case, would probably require a non-linear elastic or elastic-plastic model accounting for a reduction in stiffness with the mobilization of ground shear strength.

---

[24] This observation is consistent with three-dimensional studies published by Ward (1978). His results were made dimensionless with respect to the field stress, tunnel radius and E modulus.

Another feature that may be observed in Figure 5.9 is that for Run 2, the measured crown settlements ahead of the face are slightly smaller than those predicted. Although this may be explained by appealing to innacuracies in the field measurements, two other explanations might be given:

1. Innaccurate soil properties:

   As explained in Section 4.4.1, Chapter 4, properties of the variegated soils used in the analysis were derived from earlier studies by Sousa Pinto and Massad (1972). These authors report the occurrence of disturbance in the samples, which were obtained using Shelby tubes (Φ 12cm). Since the E modulus is sensitive to sample disturbance (i.e generally the more disturbed the sample is, the smaller the E modulus), one could speculate that the initial tangent moduli used in the analysis were smaller than the actual ones. It was not possible however to evaluate the degree of sample disturbance and its effect on the moduli.

It is also possible that the E modulus assigned for the porous clay, which was derived from back analyses presented by Negro and Eisenstein (op.cit.), is slightly low. The point being analysed is situated close to the assumed transition between the porous clay and the variegated soil.

2. Stress Paths:

It may also be argued that the explanation lies in the stress paths actually followed in the field[27]. The stress paths for several points around the tunnel obtained from the 3D analyses (Run 2) are presented in Appendix F.

If one neglects the influence of the intermediate principal stress σ_2, it is seen that the stress path followed by a point about 0.5m above the crown would be closer to that followed in an extension triaxial test, where the major principal stress σ_1 is kept constant while the minor principal stress σ_3 is decreased. Medeiros (1979) verified that these tests tend to yield higher moduli than conventional triaxial compression tests, which were used in the analysis. A 'softer' behaviour than that occurring in the field would be then predicted.

---

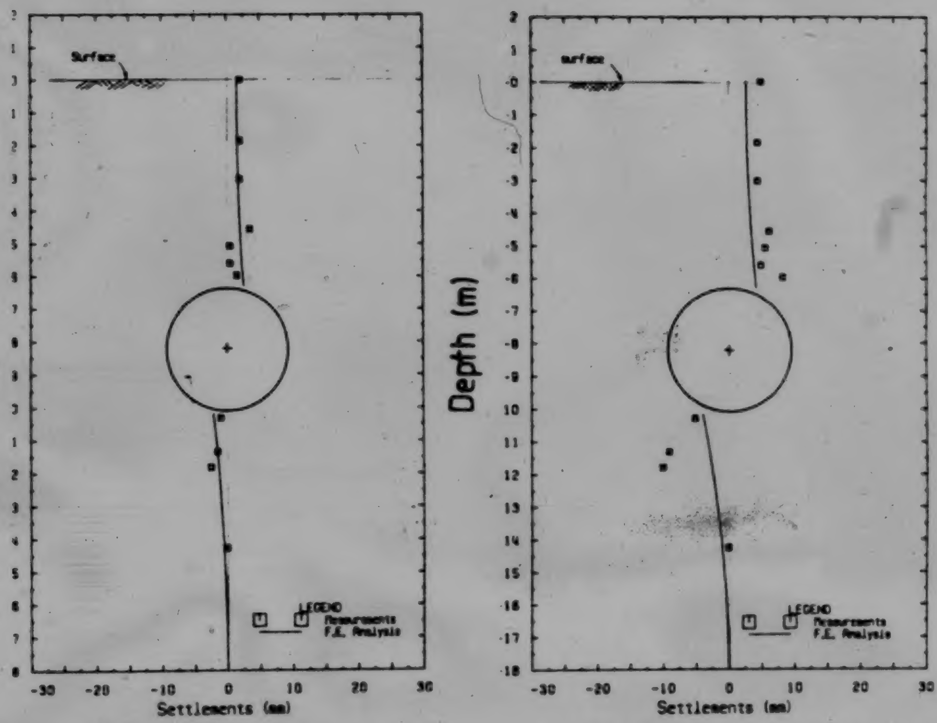
[27] The importance of using results of tests which follow the stress paths in the field has been stressed by Eisenstein and Medeiros (1983), with respect to deep excavations.

The difference between predicted and measured values is, however, minimal and since no information regarding the behaviour of the ABV site soils was available, there can be no definitive conclusion. Moreover, factors such as the influence of σ_2 or the effect of the rotation of principal stresses on the deformation properties of the ABV soils could not be assessed.

5.2.3.3 Transverse Settlement Profile

Further illustration of the relative accuracy of the three-dimensional finite element analyses is presented in Figure 5.11. Transverse profiles of the subsurface settlements predicted and measured show again that the overall behaviour before the tunnel face reaches the test section is closely predicted by a linear elastic model. The final profile, however, is not satisfactory, suggesting that the behaviour in the field assumed a non-linear mode after the face passed.

It is interesting to recall that the simpler two-dimensional analysis of this tunnel presented in Chapter 4 yielded results which were much closer to the actual measurements. Those analyses, although using a non-linear model for the variegated soil, were also much simpler in terms of data preparation and output interpretation.

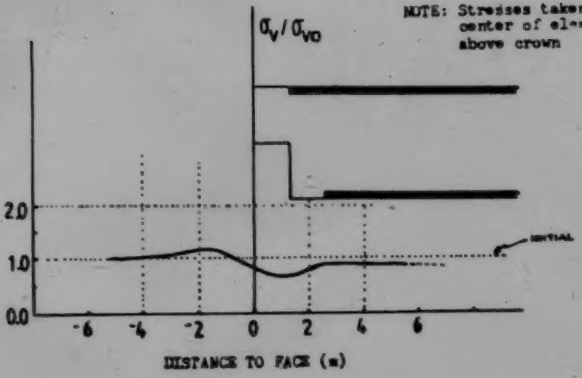


SUBSURFACE SETTLEMENTS AHEAD OF THE FACE

FINAL SUBSURFACE SETTLEMENTS

Figure 5.11 Transverse subsurface settlement profiles for ABV tunnel (only results of Run 2 are shown)

a) PRESENT STUDY (RUN 1)



NOTE: Stresses taken at center of element above crown

b) SAUER & JONUSCHEIT

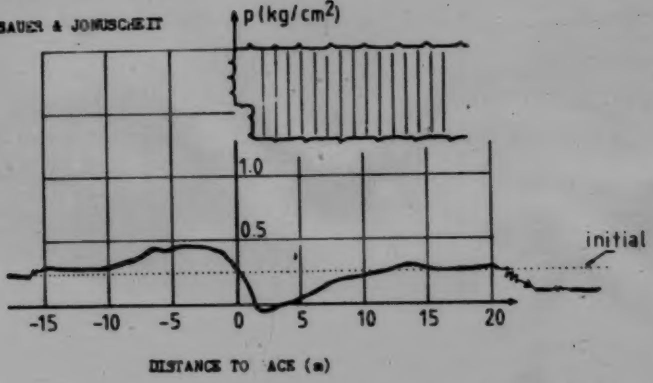


Figure 5.12 Development of vertical stresses for a point 0.5m above the crown

5.2.3.4 Vertical Stresses

Stresses were obtained at the center of the 3D elements surrounding the opening. It is possible to draw a qualitative picture of the development of the radial stresses above the crown, as shown in Figure 5.12a. It is observable that the vertical stresses ahead of the face increase at about one diameter ahead of the face. As the face passes the test section however, the stresses fall to a minimum and subsequently rise again with the installation of the lining.

This 'wavy' pattern of stress development can be explained in terms of three-dimensional load transfer mechanisms around the tunnel face, as commented in Section 5.3. For comparison, vertical pressure readings reported for a tunnel in Frankfurt (Sauer and Jonuscheit, 1976) are also shown. It is clear that the 3D analyses present results similar to those measurements.

5.2.4 Displacement Pattern with Face Advance

The following sections contain information derived from the finite element analysis (Run 2) which cannot be critically evaluated due to the absence of a complete set of field measurements for the ABV tunnel. Nevertheless, this information may be used to characterize the general displacement pattern around this specific tunnel. A certain degree of confidence in the results exists because the trends observed in the field were consistent with the results in the preceeding section.

5.2.4.1 Lateral Movements due to Face Progress

The predicted lateral movements at the test section during tunnel advance are shown in Figure 5.13. In the field, similar types of movements would be measured by an inclinometer installed at 1.50m from the tunnel. The lateral movements are shown for situations where the face of the tunnel is 7.80m (2 diameters) and 2.60m before the reference section, at the test section, and past it by 3.25m and 5.60m respectively.

As the tunnel approaches the reference line, the lateral movements are towards the tunnel. From Figure 5.13 it can be seen that the onset of these displacements occurs when the face is at approximately 1.5-2.0 diameters from the reference section. After the face passes, the movements increase even more until the lining is installed. It is interesting to note that a point at the surface, which is free to move, does not displace significantly in the horizontal direction. This is an indication that movements at the surface are largely vertical. A similar pattern of behaviour has previously been observed for shield driven tunnels (e.g. Branco, 1981).

5.2.4.2 Longitudinal Axial Movements

Longitudinal axial movements could be measured in the field if an inclinometer casing was installed along the centerline of the tunnel and measurements taken as the face approached. The calculated movements are shown in Figure 5.14. It is observed that when the face is at 7.80m (2 diameters) the movements are barely noticeable. As the face approaches, the movements are always towards the face with the amount increasing as the tunnel comes closer to the reference section. A tendency of the soil to move towards the excavated heading is also noticeable. As in the case of the lateral movements, a point at the surface remains almost stationary, suggesting that the movements at the surface are largely vertical while near the tunnel the displacement vectors 'flow' towards the face. This is indeed verified by inspection of the displacement pattern in Figure 5.16.

5.2.4.3 Longitudinal Movements at Side of Tunnel

Figure 5.15 depicts longitudinal movements at the side of the tunnel, which would be measured in the field by an inclinometer installed at the side of the tunnel. At the location analysed (1.50m from tunnel springline) the movements are less significant than those at the axis (it should be noted that the horizontal scale is different than that used in Figure 5.14.).

The longitudinal horizontal movements are still predominantly in the face direction and increase in

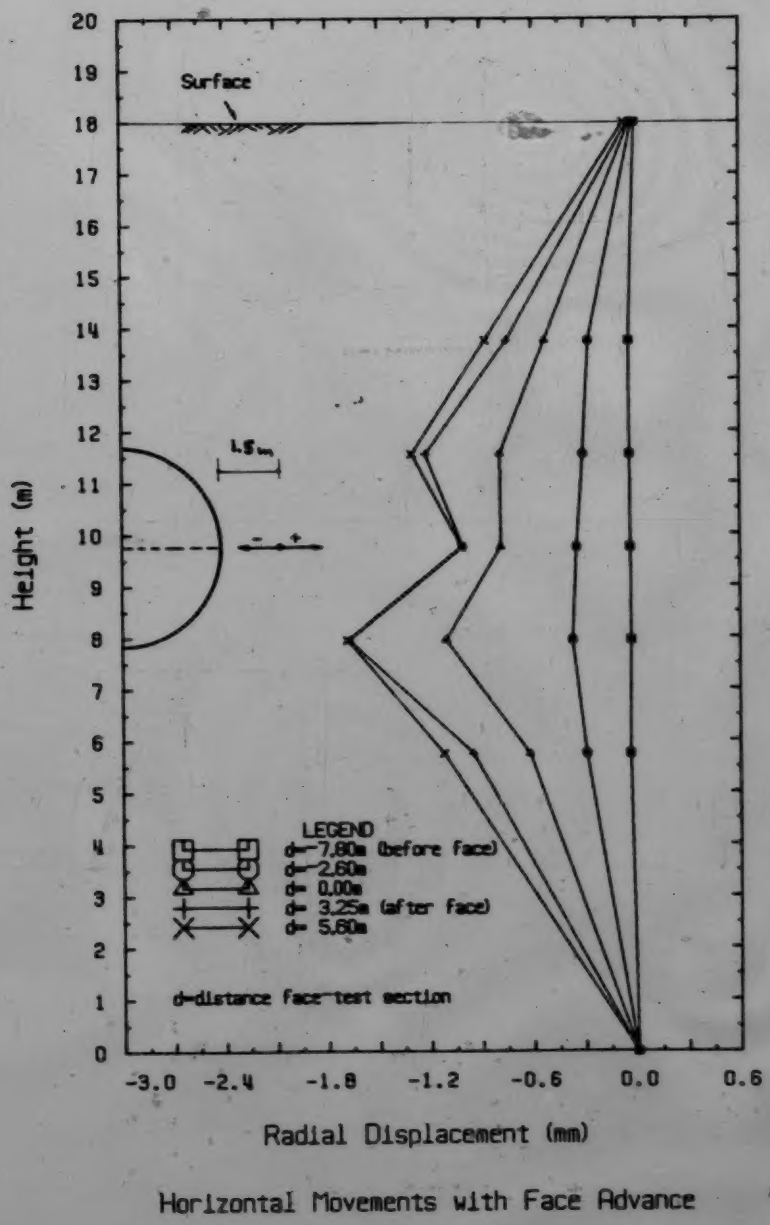


Figure 5.13 Lateral movement at test section – ABV tunnel

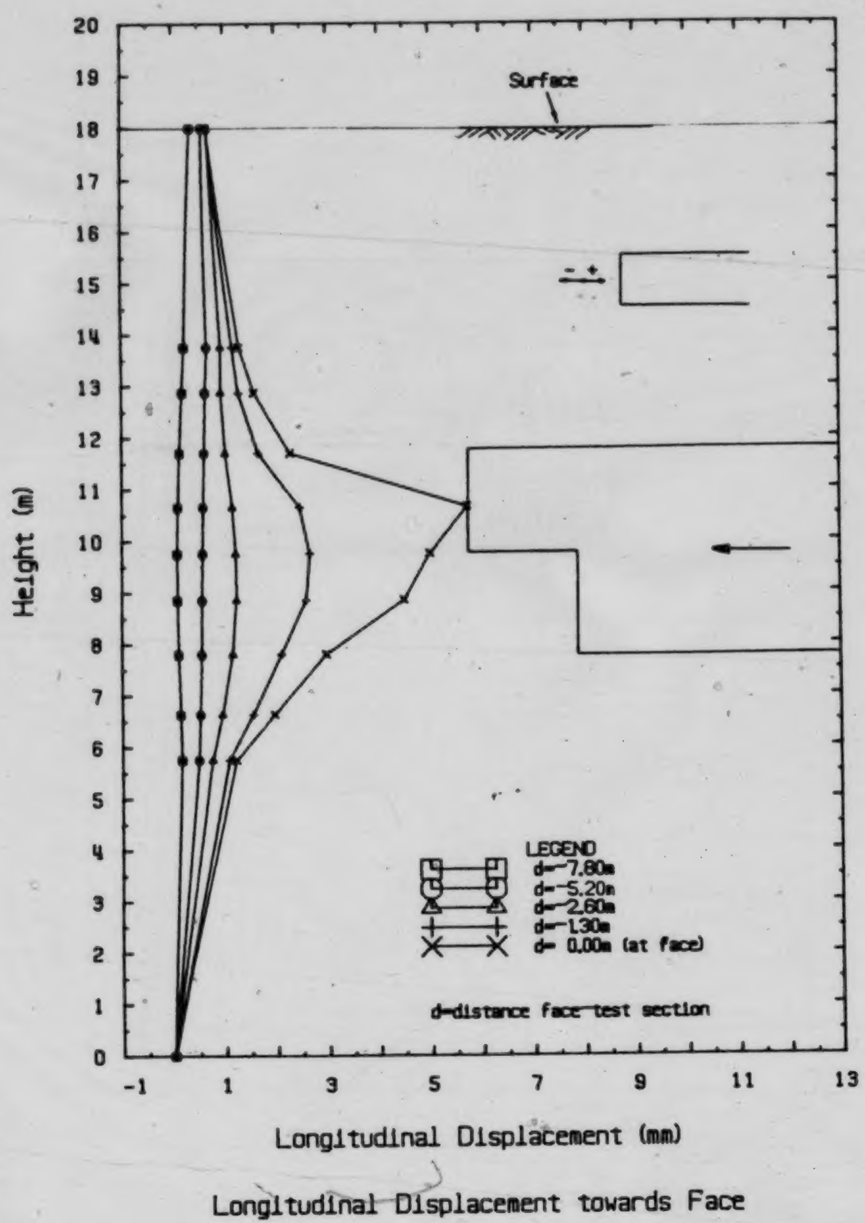
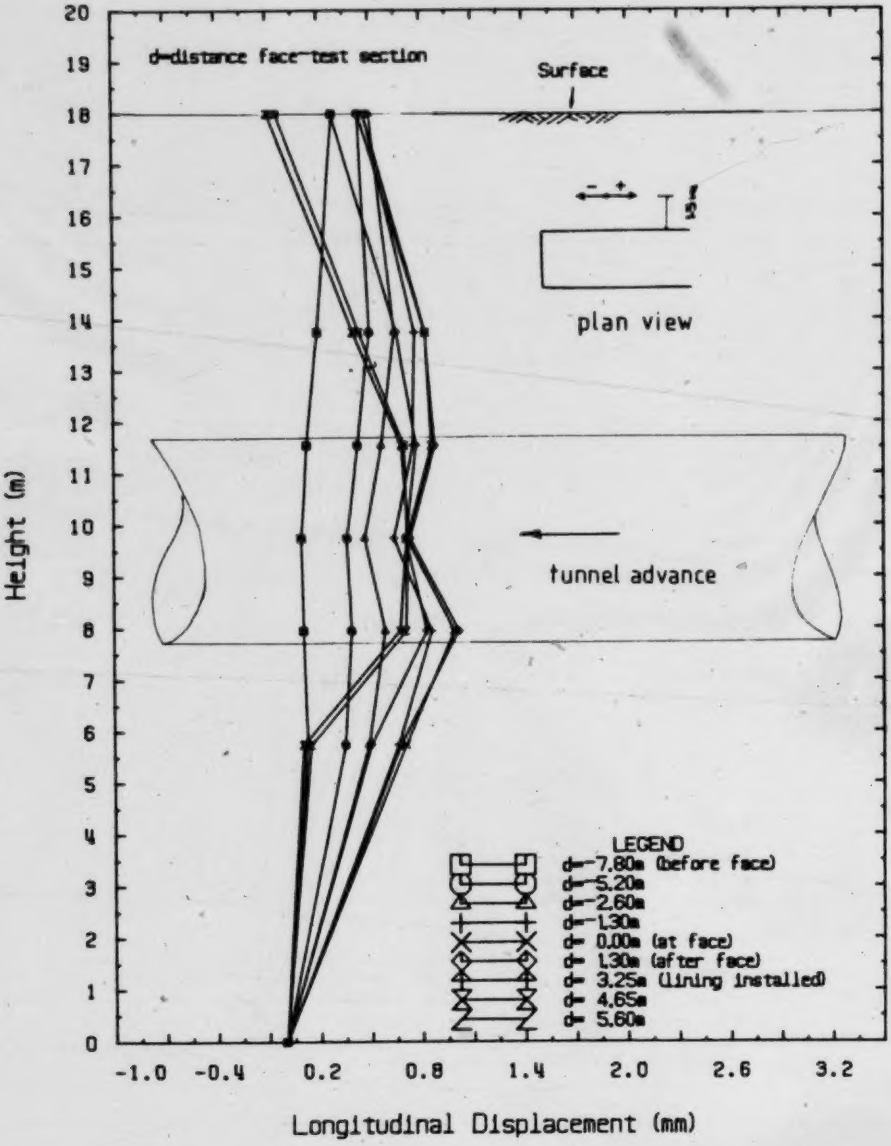


Figure 5.14 Longitudinal displacements towards the face: ABV tunnel



HORIZONTAL MOVEMENTS WITH FACE ADVANCE

Figure 5.15 Longitudinal movements for a point located 1.5m from springline: ABV tunnel

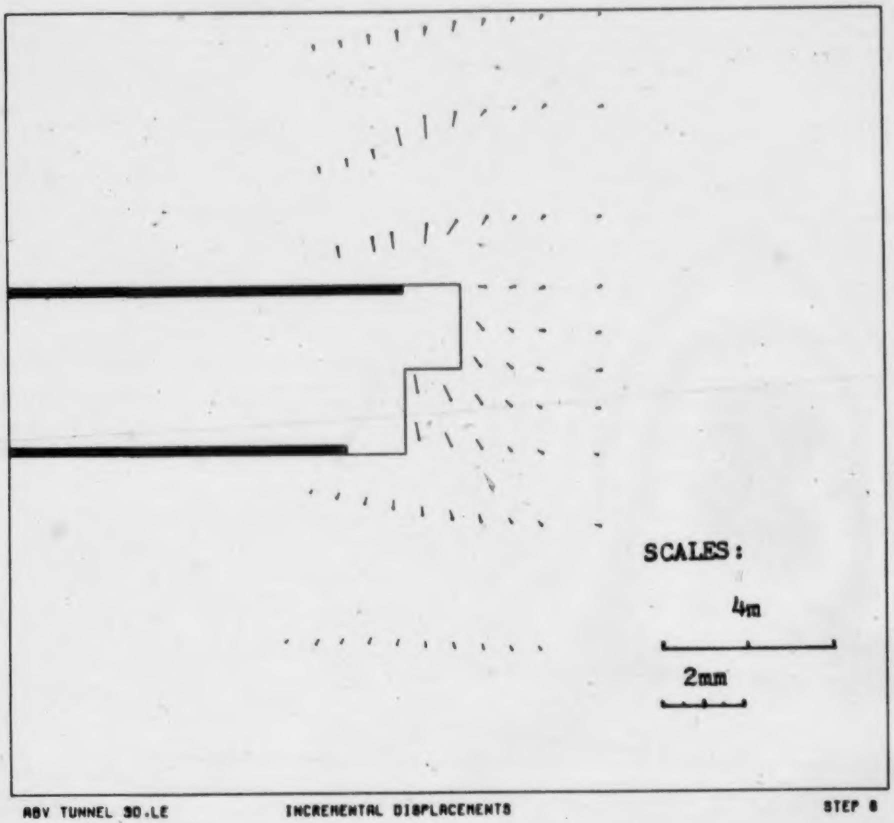

SCALES:
4m
2mm
ABV TUNNEL 3D.LE
INCREMENTAL DISPLACEMENTS
STEP 8

Figure 5.16 Distribution of displacements at tunnel centre-line due to one excavation step: ABV tunnel

magnitude as the tunnel advances. After the face passes the movements show a slight trend for the soil to move back to its original position. This effect has been observed in previous finite element analyses (Ranken and Ghaboussi, 1975) and also in the field, for the case of shield driven tunnels (Branco, 1981:112), but no satisfactory explanation has been offered so far. It is apparent that these 'backward' movements around the tunnel are due to the 'elastic rebound' of the tunnel face against the direction of excavation. That is, since points within the tunnel perimeter are displacing into the tunnel, the points in the vicinity of the tunnel will move backwards in order to compensate for the volume lost through the face.

The fact that there is some longitudinal movement in areas close to the contact lining-soil in NATM tunnels has not been reported in any of the references assessed. This suggests that analytical or numerical studies based on the assumption of plane strain cannot fully reflect reality. Field measurements of longitudinal movements close to the lining-soil contact are urgently needed to clarify these points.

5.2.4.4 Movements at Symmetry Plane

In order to examine more closely the effect of the lining erection on the displacement pattern, it is interesting to investigate the displacements generated by one step of tunnel excavation. These displacements are depicted in Figure 5.16, which shows that the displacements ahead of the tunnel are largely towards the face, assuming a more vertical pattern above and below the unsupported cavity. After lining installation however, there is a tendency of the points around the tunnel to move back towards the direction of excavation, as explained in the preceding section. Again, this enhances possible limitations of plane strain models in analyses of soil-lining interaction close to the tunnel face.

5.3 Stresses and Displacements near the Face

5.3.1 Outline of the Problem

Excavation of a tunnel provokes significant changes in the in-situ stress field. If the tunnel is viewed longitudinaly, three distinct regions can be identified (Ranken and Ghaboussi, 1975:1-1):

1. **Far ahead of the face:** This zone has not been disturbed by excavation and the in-situ stresses govern. If one principal stress is vertical and the two horizontal principal stresses are of equal magnitude, the in-situ stress field may be represented by a two-dimensional model.
2. **Around the face:** In this zone, the load transfer mechanisms generated by tunnel excavation create a three-dimensional pattern of stresses and displacements

which cannot be represented by a two-dimensional model.

3. **Well behind the face:** The stress displacement pattern will normally return to a two-dimensional condition.

In elastic ground, the extent of the three-dimensional zone around the face is known to be one to two diameters ahead of and behind the face (Ranken and Ghaboussi, 1975:5-1). When plastic yielding occurs, the extent of the 3D zone increases, mainly behind the face. It is normally in this zone that the support of an NATM soft ground tunnel will be placed[28], due to the requirement of minimizing ground movements. In time independent materials, the development of lining loads will take place entirely within this 3D zone, as explained in the following section.

5.3.2 Load Transfer Mechanisms around the Face

The evolution of lining pressures occurring during tunnel advance can be explained by the three-dimensional arching mechanisms postulated by Eisenstein et al. (1984) and illustrated in Figure 5.17[29]. As excavation progresses, the removal of ground up to the new face causes a load redistribution around the tunnel. Some of the redistributed load will be transfered to the support and some to the unexcavated ground ahead of the face. At some point back from the face, the ground stresses stabilize in equilibrium with the lining.

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[28] This is probably the case with most shield tunnels as well.

[29] Only the longitudinal arching is studied herein.

Eisenstein et al. (op.cit.) present a conceptual attempt to relate the radial stresses and displacements for a point at the tunnel crown. Figure 5.18 shows the vertical stress and displacement distribution along a longitudinal line at the tunnel crown, as well as the displacements which occur along the same line. It is argued that there should be a stress concentration ahead of the face (point B), followed by a rapid decrease of vertical stress to zero at the tunnel face (point C). The stress is zero along the unsupported cavity (points C-D), provided that no internal pressure is applied. With the lining installed, there is a new increase in vertical stress and stable equilibrium is eventually reached at point F. The stress concentration near the lining leading edge (point E) has been shown to depend on the relative stiffnesses of the lining and of the ground (Schwartz and Einstein, 1980:72). If the support is very stiff with respect to the ground, this stress concentration will be present.

Combining the stress and displacement distribution as shown in Figure 5.18a it is possible to draw a ground response curve similar to that postulated in Chapter 1, but including features that more realistically reflect the actual stress and displacement patterns. This curve is shown in Figure 5.18b and , although conceptual, it has the merit of recognizing the actual load transfer mechanisms existing at the tunnel face.

5.3.3 Field Evidence

Field evidence of this stress redistribution pattern around the face has been presented in the case history at the Frankfurt "S-Bahn - Los 6" tunnels described in Appendix A. The vertical pressure measured within the ground mass at the crown of one of the tunnels has been reproduced in Figure 5.12. Regarding the stress concentration near the lining leading edge, no published field evidence has been found by the Author, at least in the case of NATM tunnels.

5.3.4 Numerical Analyses

Most previous numerical studies of the behaviour near the face made use of axisymmetric models (e.g. Ranken and Ghaboussi, 1975). The results of these analyses are strictly valid for the case of a deep tunnel where the in-situ stress field is uniform (i.e., $K=1$). The stress-displacement behaviour is thus the same for crown, springline and invert. In order to investigate the development of stresses and displacements around a shallow tunnel and with $K\neq1$, truly three-dimensional studies are necessary.

The following results are derived from 3D analyses termed Run 3 and Run 4. The mesh used is shown in Figure 5.19. It should be noted that this mesh is a modification of the mesh presented in Figure 5.5. In order reduce the costs, two slices were eliminated by reducing the longitudinal dimensions (see Figure 5.19). Selection of the material properties was arbitrary. The sequence of excavation is as

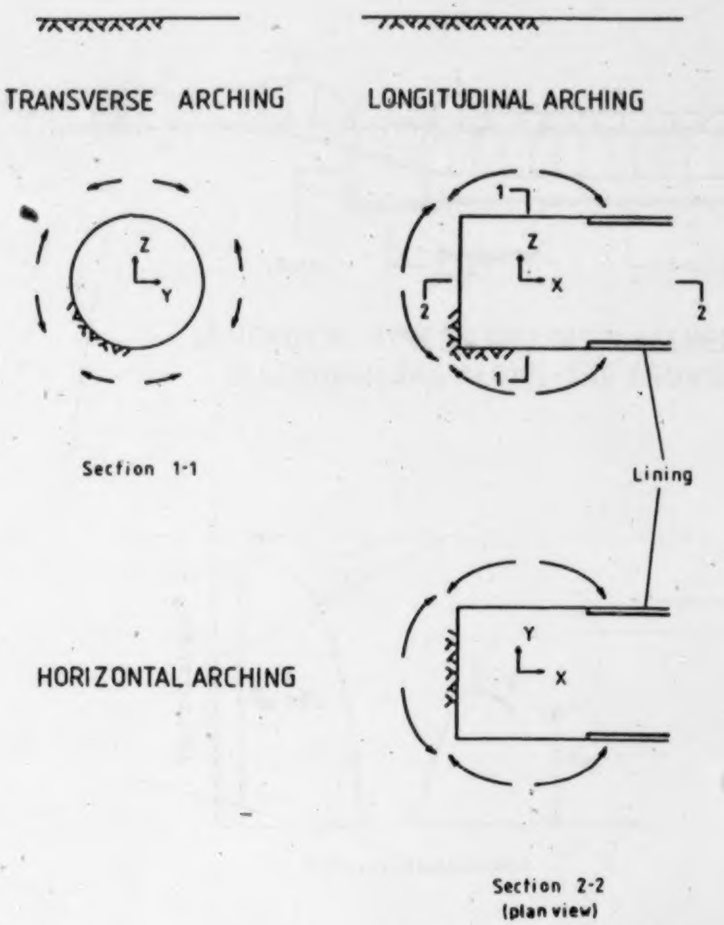
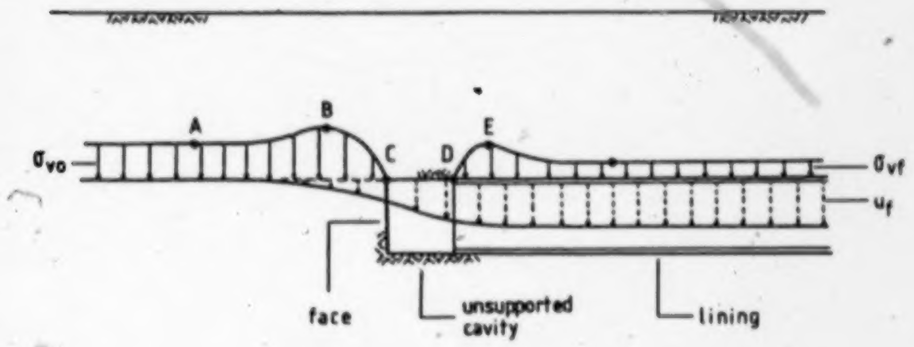
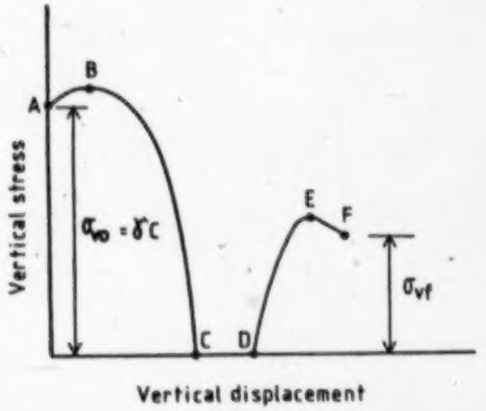

TRANSVERSE ARCHING
LONGITUDINAL ARCHING
Z
Y
1
Z
X
2
2
1
Section 1-1
Lining
Y
X
HORIZONTAL ARCHING
Section 2-2
(plan view)

Figure 5.17 Three-dimensional arching around an advancing tunnel (after Eisenstein et al. 1984:modified)


A
B
C
D
E
σvo
σvf
uf
face
unsupported cavity
lining

a) VERTICAL STRESS AND DISPLACEMENT DISTRIBUTIONS ALONG THE CROWN



b) GROUND RESPONSE AT THE CROWN

Figure 5.18 Conceptual ground response at the crown (after Eisenstein et al., 1984:modified)

follows:

Step 1: Establishment of initial conditions (i.e, application of gravity)

Step 2: Full face excavation of slice 1

Step 3: Full face excavation of slice 2

Step 4: Full face excavation of slices 3, 4 and 5

Step 5: Full face excavation of slices 6 and 7

Step 6: Full face excavation of slices 8 and 9

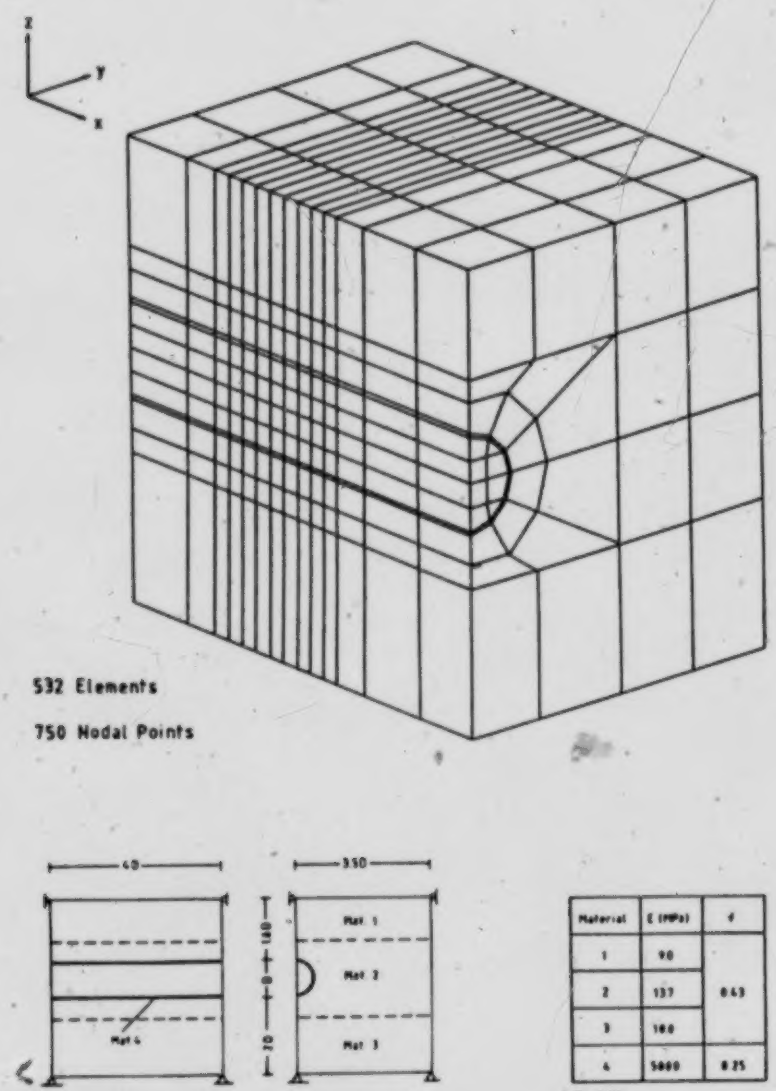
Step 7: Full face excavation of slices 10 and 11

Step 8: Full face excavation of slices 12 and 13

Step 9: Full face excavation of slice 14

It is important to observe that in the following sections, the stresses refer to points located approximately 20cm from the opening, while the displacements are obtained for nodes at the excavation line. Ideally, the same point should be referred to. This was not done because the stresses at the nodes are known to be innaccurate (Bathe, 1982:177).

Figures 5.20 and 5.21 show the computed results for radial stress and displacement distributions at the crown, for two distances of lining installation behind the face (L=(1/2)D and L=(2/3)D). Due to numerical innacuracies, as discussed at the end of this chapter, there is a considerable scatter of stresses and displacements behind the advancing face. For this reason, the curves have been smoothed out after superimposing output stresses and displacements from three consecutive steps of face advance


z
y
x
532 Elements
750 Nodal Points
40
350
180
80
70
Mat 1
Mat 2
Mat 3
Mat 4

Material	E (MPa)	ν
1	90	
2	137	0.43
3	180	
4	5000	0.25

Ko=0.75

Figure 5.19 3D Mesh used in Runs 3 and 4 (this was done by making the face position in these three steps coincide).

It can be observed that the average stress distributions did not show any significant stress concentration ahead or behind the unsupported cavity. However, it is evident that the radial stresses decrease sharply from their original values for points immediately ahead of the face. They did not drop to zero as postulated in Figure 5.18, because they were not taken at a point at the excavated boundary, as commented above. It should also be noted that this point starts to partly recover stress at some distance ahead of the lining leading edge.

5.3.5 Ground Response Curves

By combining radial stresses and displacements at corresponding points in Figures 5.20 and 5.21, one may draw ground response curves. These curves, presented in Figure 5.22, are defined as a relationship between radial stress and displacement for a point at the tunnel perimeter. It should be noted that in Figure 5.22 the stresses and displacements have been normalized to the in-situ vertical stress and to the tunnel radius respectively.

Both ground response curves show similar trends. It is noticeable that at an earlier stage, displacements begin to occur without a major vertical stress change. As the face approaches the reference section, a faster drop in the stresses occurs. Ideally, the stresses at the perimeter of

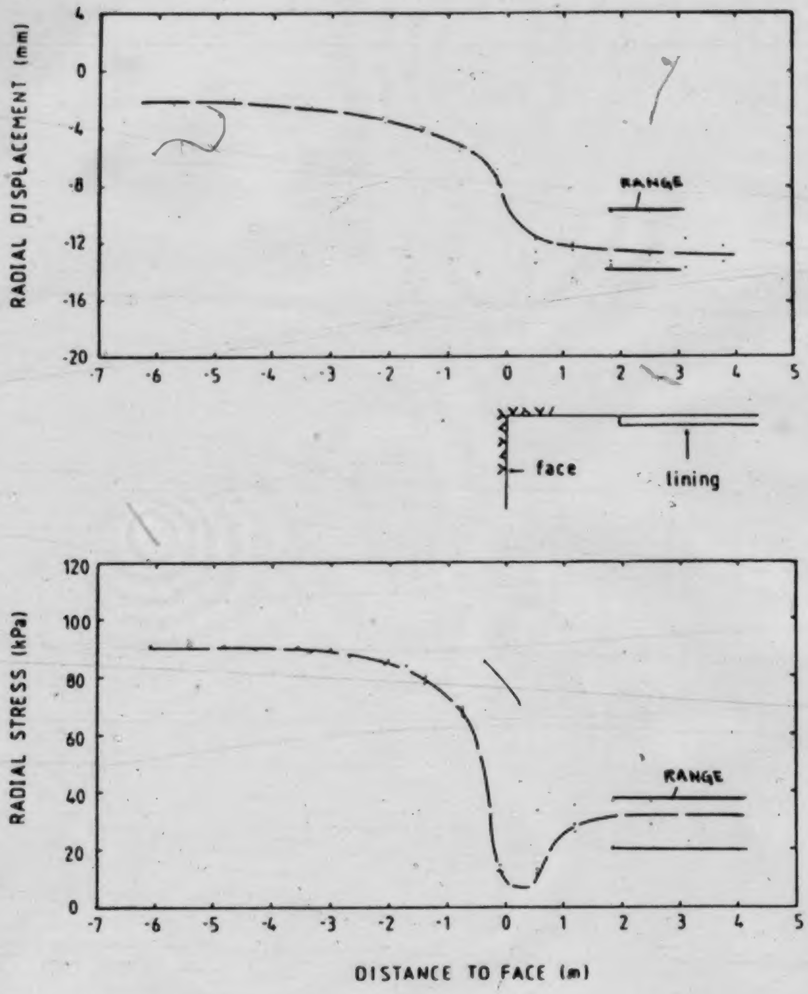


Figure 5.20 Stresses and displacements at crown: L=(1/2)D

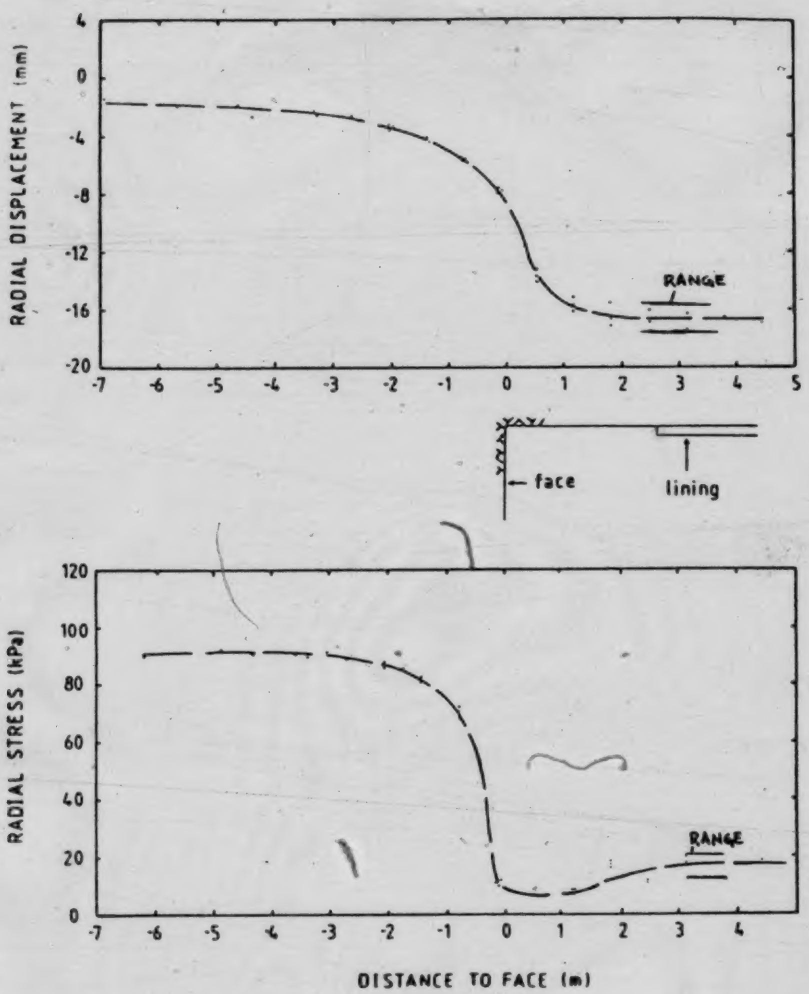


Figure 5.21 Stresses and displacements at crown: L=(2/3)D

the unsupported tunnel should be zero as the face passes through the reference section. They do not drop to zero because they refer to a point not at the excavation line but at some small distance above it.

Even if the stresses are not zero, it is noticeable that at this stage they remain fairly constant, while the displacements increase further. As the leading edge of the lining moves forward, the stresses increase until an equilibrium situation is finally achieved.

These results clearly show that loading on the support is dependent on how close the support is installed to the face. The support closest to the face carries more load. As expected then, the final displacements calculated for the case where the lining is installed close to the face are smaller than for the further case. These observations support the concept of ground and support characteristic curves presented in Chapter 1.

Another important observation is the fact that the three-dimensional nature of the problem accounts for a non-linear ground response curve, even in a linearly elastic material.

5.3.6 Behaviour at Springline and Invert

The 3D arching mechanisms shown in Figure 5.23 allow the postulate of similar stress patterns as those previously described, for the invert and for the springline. Again, the same remarks regarding the existence of stress

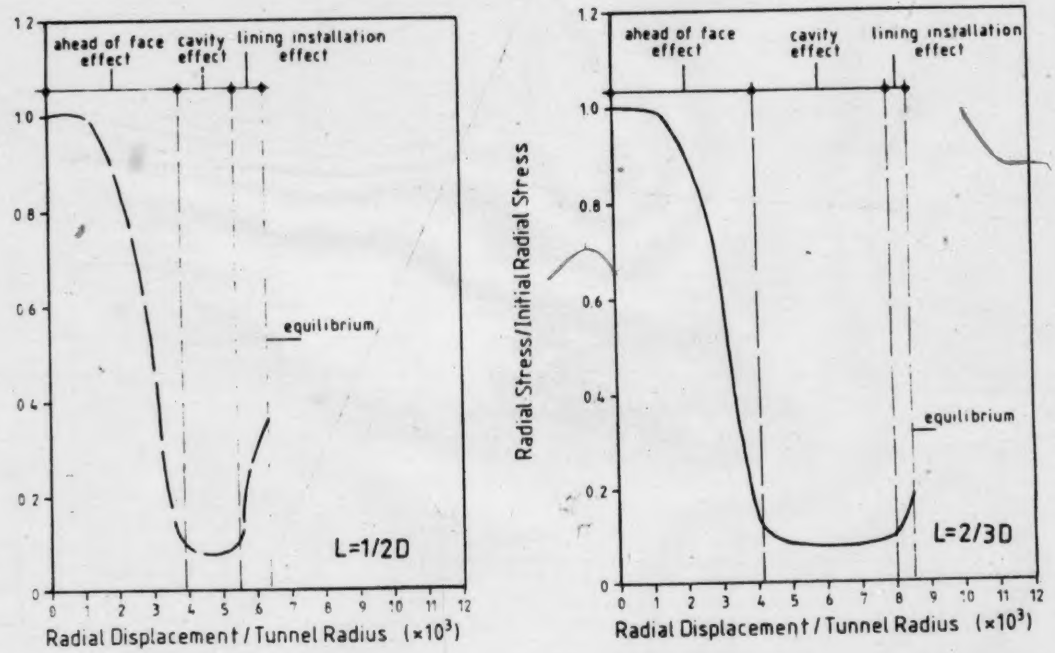


Figure 5.22 Ground response curves at the crown

concentrations ahead or behind the unsupported cavity made in Section 5.3.2 are applicable.

Figures 5.23 to 5.26 show the results of the analyses for radial stress and displacement distribution along the invert and the springline, for the two distances of lining installation behind the face. The curves have been obtained by the same procedure described in Section 5.3.4.

It should be noted that the displacement development at the invert (Figures 5.26 and 5.27) is not consistent with the pattern observed for the crown, presented in Figures 5.20 and 5.21. These invert displacements are consistently inwards ahead of the tunnel and around the unsupported cavity. After the lining installation however, the points at the invert tend to move outwards in the direction of their original position. This is also verified for the springline, but only in the case where the lining is installed close to the face (L=(1/2)D). No evidence of such behaviour was found in the case histories investigated. However, in most of the cases analysed the displacements near the tunnel were measured at points located at a certain distance from the tunnel boundary. It is possible that this behaviour could indeed occur for points at the lining-soil contact.

5.3.7 Evaluation of Final Loads and Displacements

The determination of the final equilibrium stresses and displacements behind the face and after the lining installation using three-dimensional finite element analyses

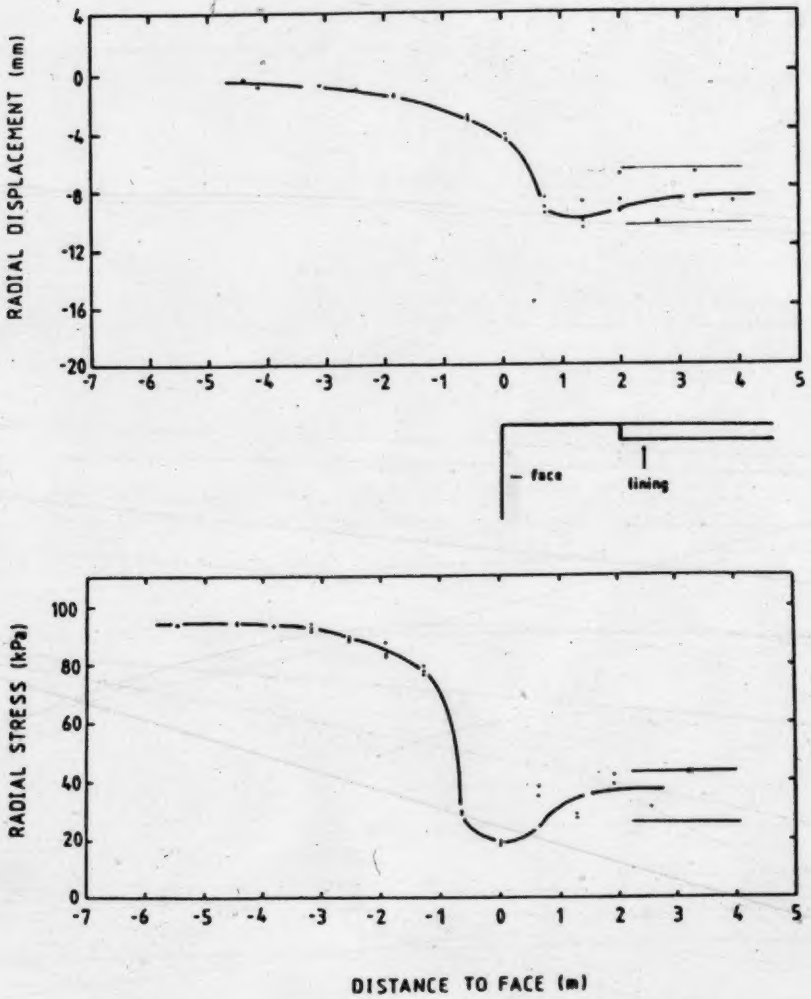


Figure 5.23 Stresses and displacements at springline: L=(1/2)D

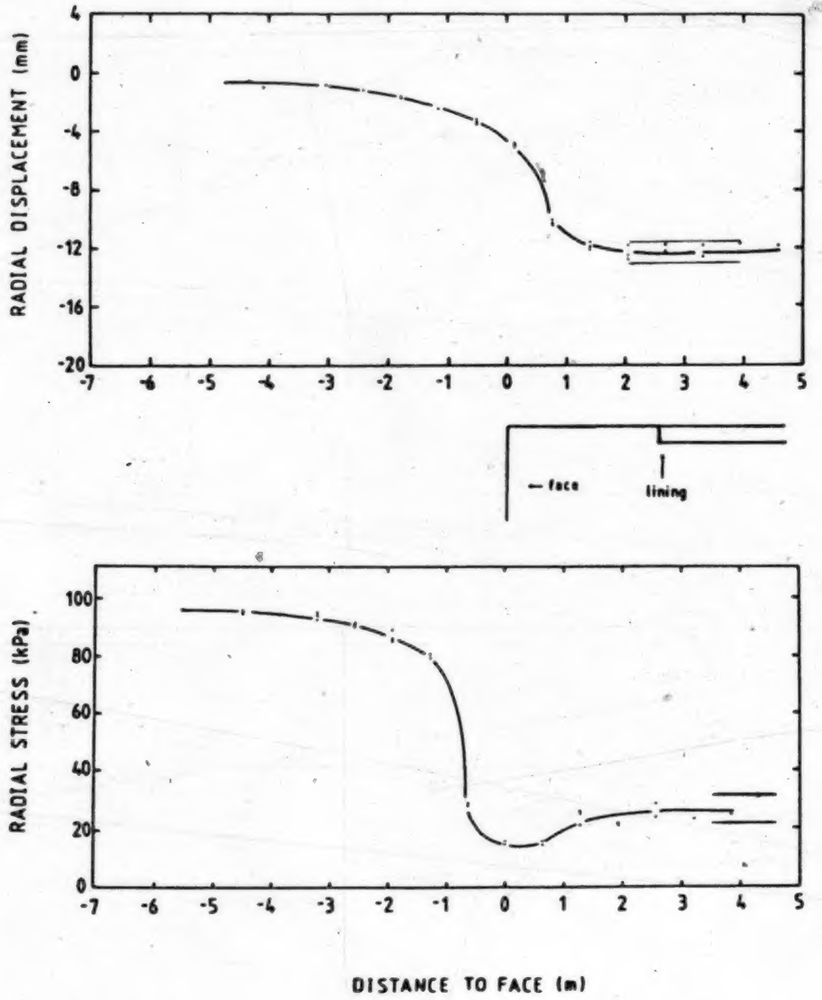


Figure 5.24 Stresses and displacements at springline: L=(2/3)D

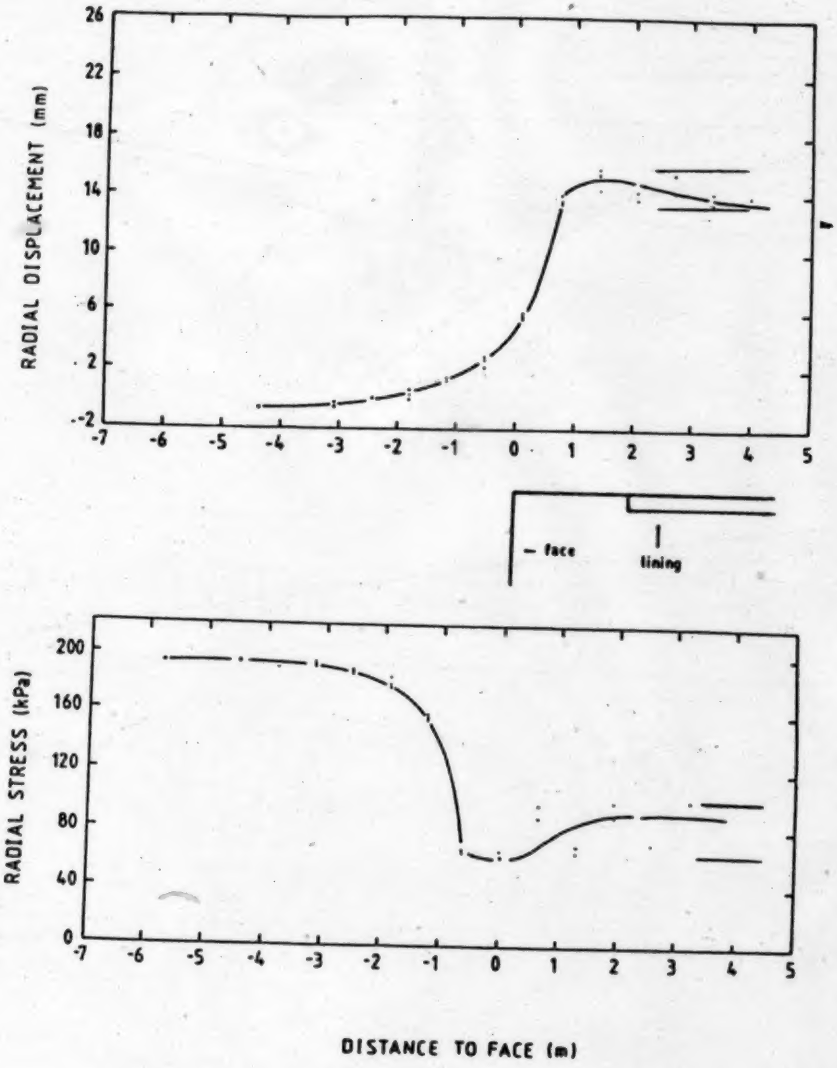


Figure 5.25 Stresses and displacements at invert: L=(1/2)D

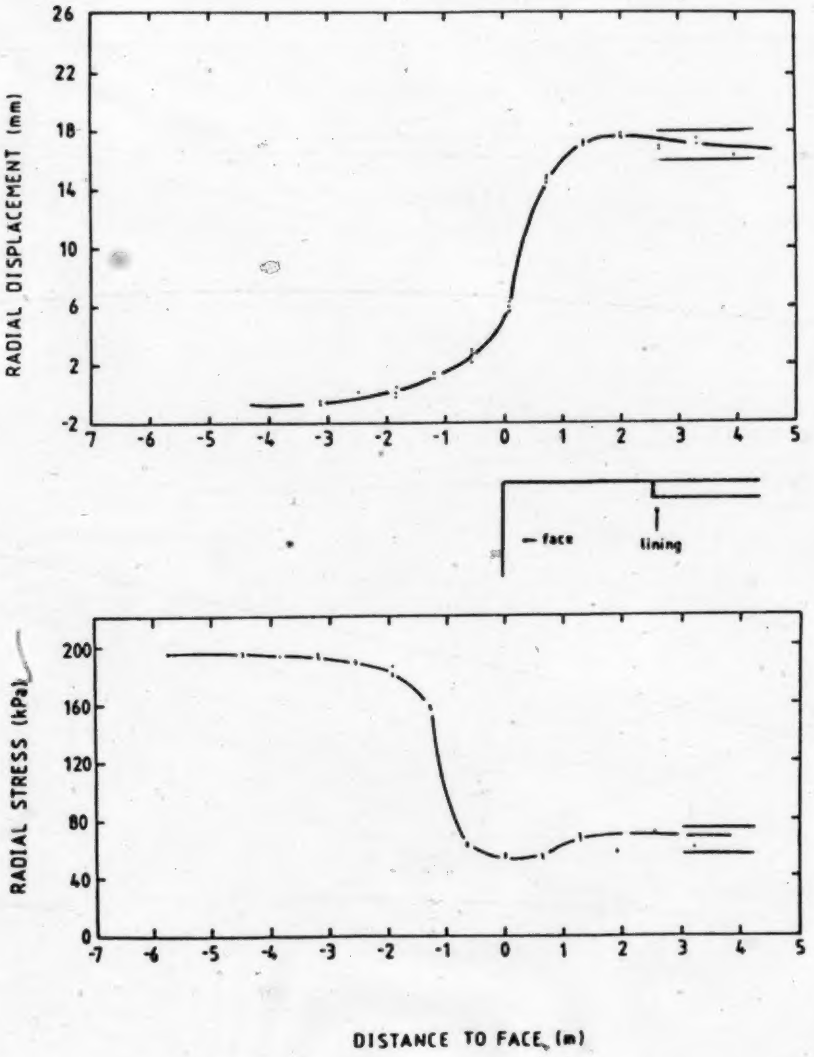


Figure 5.26 Stresses and displacements at invert: L=(2/3)D

is still an expensive and time consuming procedure to be used on a case by case basis. For this reason, two–dimensional simplified procedures which represent the behaviour well behind the tunnel face, accounting for a reduction in pressures according to the concept of characteristic curves introduced in Chapter 1 have been developed.

The simplified procedures analysed herein are based on the concept of representing the ground and support behaviour through these characteristic curves, which is sometimes termed the 'Convergence Confinement Method' (this concept was introduced in Section 1.4.1, Chapter 1).

5.3.7.1 Determination of the Ground Response Curve (GRC)

Several analytical procedures are available to determine the ground response curves for a variety of material behaviours, as reviewed by Brown et al. (1983). Most of these solutions however, assume total axisymmetry and a hydrostatic in–situ stress field. They are rigorously applicable only for deep tunnels where the ratio between horizontal and vertical stresses (K) is equal to one.

In ground behaving linearly elastic, the ground response curve given by these analytical procedures has been shown to be a straight line (Kaiser, 1982). For the particular case of fully elastic behaviour, it is also possible to draw the ground response curves for non–axisymmetric situations, where each point at the tunnel boundary will behave differently. This is done by using plane strain elastic solutions for a tunnel excavated in a prestressed medium (e.g. Pender, 1980).

Ground response curves representing the condition of $K<1$ are analysed in the following sections and illustrated in Figure 5.27a. It should be noted that the curves for crown and springline define a range of GRCs for the points around the tunnel. The derivation of the equation shown in Figure 5.27a is presented in Appendix E.

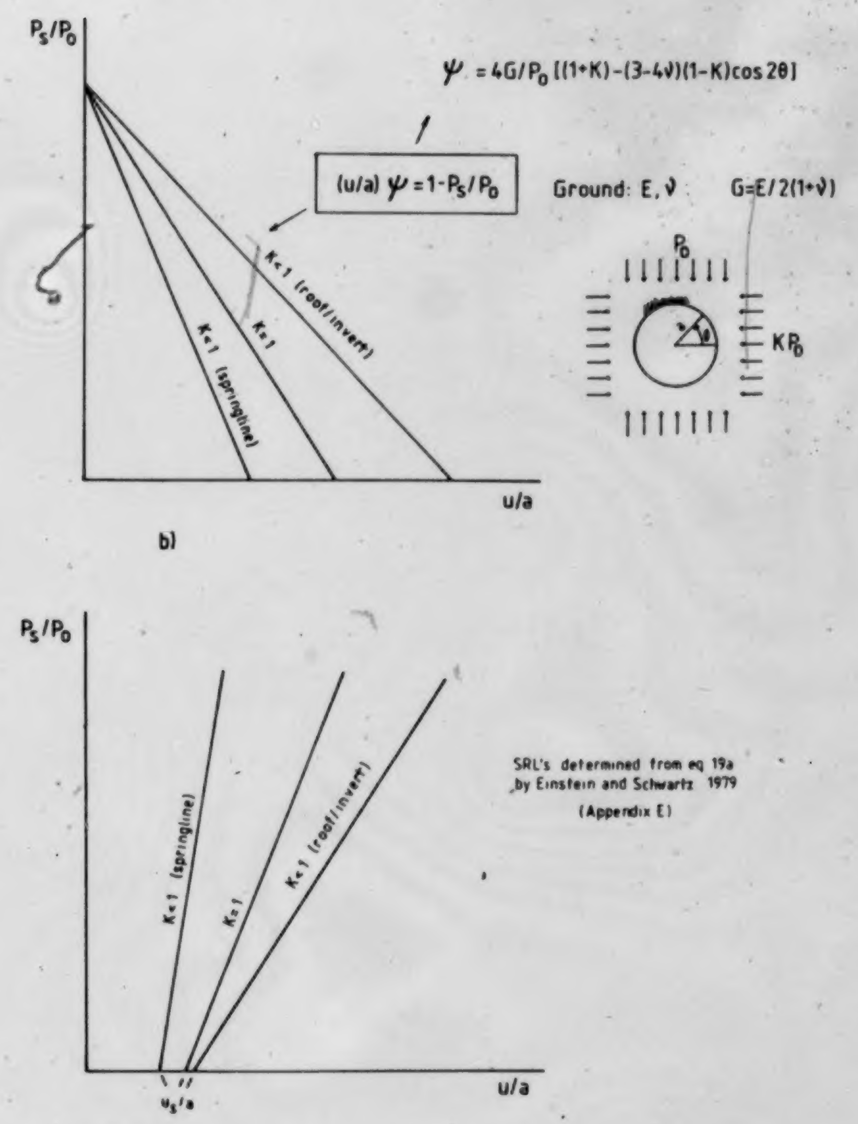
Another way of determining the ground response curve is by using two-dimensional finite element analyses. The use of the FEM enables a more rigorous analysis of situations which are not axisymmetric, as in the case of a shallow tunnel.

5.3.7.2 Determination of the Support Reaction Line (SRL)

The SRL for shotcrete is normally determined on the assumption of instantaneous hardening of the shotcrete and elastic behaviour. For $K\neq1$ situations, formulae presented by Einstein and Schwartz (1979) may be used. The derivation for the present study is presented in Appendix E.

5.3.7.3 Displacements before Support Installation

A fundamental parameter for application of the method is the displacement occurring before the support is installed (u_o/a in Figure 5.27b). Numerical studies


P_s/P_o
$\psi = 4G/P_0\,[(1+K)-(3-4\nu)(1-K)\cos 2\theta]$
$(u/a)\,\psi = 1-P_s/P_0$
Ground: E, ν
$G=E/2(1+\nu)$
P_0
KP_0
K<1 (roof/invert)
K=1
K<1 (springline)
u/a
b)
P_s/P_o
K<1 (springline)
K=1
K<1 (roof/invert)
u/a
u_s/a

SRL's determined from eq 19a by Einstein and Schwartz 1979

(Appendix E)

Figure 5.27 Characteristic curves in elastic materials

carried out using axisymmetric models (e.g. Ranken and Ghaboussi, 1975) allow the determination of u_i for $K=1$ conditions. Results from three-dimensional numerical studies of unlined tunnels presented by Ward (1978) include the $K \neq 1$ condition. The results by Ranken and Ghaboussi (op.cit.) indicate that for linings installed at more than one tunnel radius behind the face, the displacement occurring ahead of the face is independent of whether the tunnel is lined or unlined.

Hutchinson (1982:86) points out the difficulties related to determination of u_i in lined tunnels. In the case of the NATM, where the support is normally placed in stages due to the heading and bench procedure, determination of u_i is especially difficult.

For this reason, the values of u_i used in the present study are taken as the displacements occurring ahead of the face (u_0). This is a simple and conservative assumption, but reasonable because in NATM urban tunnels the support will probably be placed as close as possible to the tunnel face to minimize ground displacements. The u_0 values used to draw the support characteristic curves in the following sections are those determined from the 3D analyses.

A comparison with the displacements determined by using results from Ward (op.cit.) and Ranken and Ghaboussi (op.cit.) is made in Table 5.1. An analysis of the values shown in this table shows that:

1. Previous numerical solutions tend to overestimate the face displacement.
2. A close agreement was obtained for the springline using the results by Ward (op.cit.).
3. The K=1 solutions for the springline yield values of u_0 which are highly on the unsafe side.
4. The simplified assumption that u_0 is one third of the final elastic wall displacement of the excavated plate problem ($u=P_0a/2G$) is also on the unsafe side.

5.3.7.4 Application to the Present Study

Several studies have been published in which simplified procedures for calculating the loads and deformations of tunnel linings are checked against results of case histories (e.g. Schwartz and Einstein, 1980). The performance of such calculations requires evaluation of properties of the soil mass not readily determinable and the margin of uncertainty in these studies tends to be large.

The results of the present 3D finite element analyses, despite numerical innaccuracies, provide a better oportunity to check the validity of simplified procedures. Uncertainties regarding ground and support properties are totally eliminated, since they are part of the program input.

The final equilibrium stresses and displacements for crown, springline and invert determined in the 3D analyses and shown in Figures 5.20, 5.21 and 5.23 to

Table 5.1 Displacements ahead of the face

Uo/a Method	Uo/a(cr)	Uo/a(sp)	Uo/a(in)	REMARKS
3D - L=1/2D	3.8	2.4	2.9	Lin.El. K=.75
3D - L=2/3D	4.2	2.5	2.9	"
Ward (1978)	5.1	2.7	5.1	Lin.El. K=0/K=1 interp. for K=.75
Ranken/ Ghaboussi (1975)	5.3	5.3	5.3	Lin.El. K=1 unlined
1/6 Po/G	5.7	5.7	5.7	G=E/2(1+ν)

Note: Uo = radial displacement ahead of the face (×1000)
a = 1.95m, Po=165kPa, E=13.7MPa, ν=0.43

5.26 are shown in Figures 5.28, 5.29 and 5.30. Because on the numerical uncertainties, a range of calculated values is presented, as shown in the previous figures.

In the same figures are the results of calculations using the closed form solutions derived previously, as well as results from a two–dimensional finite element analysis. The mesh and material properties used in this analysis correspond to a plane strain section of the 3D mesh (Figure 5.19). The excavation was carried out in one step only and no lining was installed. As in the 3D analysis, the stresses refer to points located approximately 20cm from the opening, while the displacements are taken at the excavation line.

An initial important observation that can be made from Figures 5.28 to 5.30 is that the equilibrium points at the springline fall very close to the GRC generated by the 2D finite element analysis. This is not verified at the crown of the tunnel. The explanation for this is believed to be the problem of the location of the bottom rigid boundary studied in Chapter 4. The E modulus was kept constant throughout the opening region for easier comparison with the closed form solution, as shown in Figure 5.19. In 2D analyses the displacements are very sensitive to this assumption, which tends to underestimate the heave at the invert and overestimate the crown displacement. The springline is not affected much. In the 3D case, the problem of a constant modulus

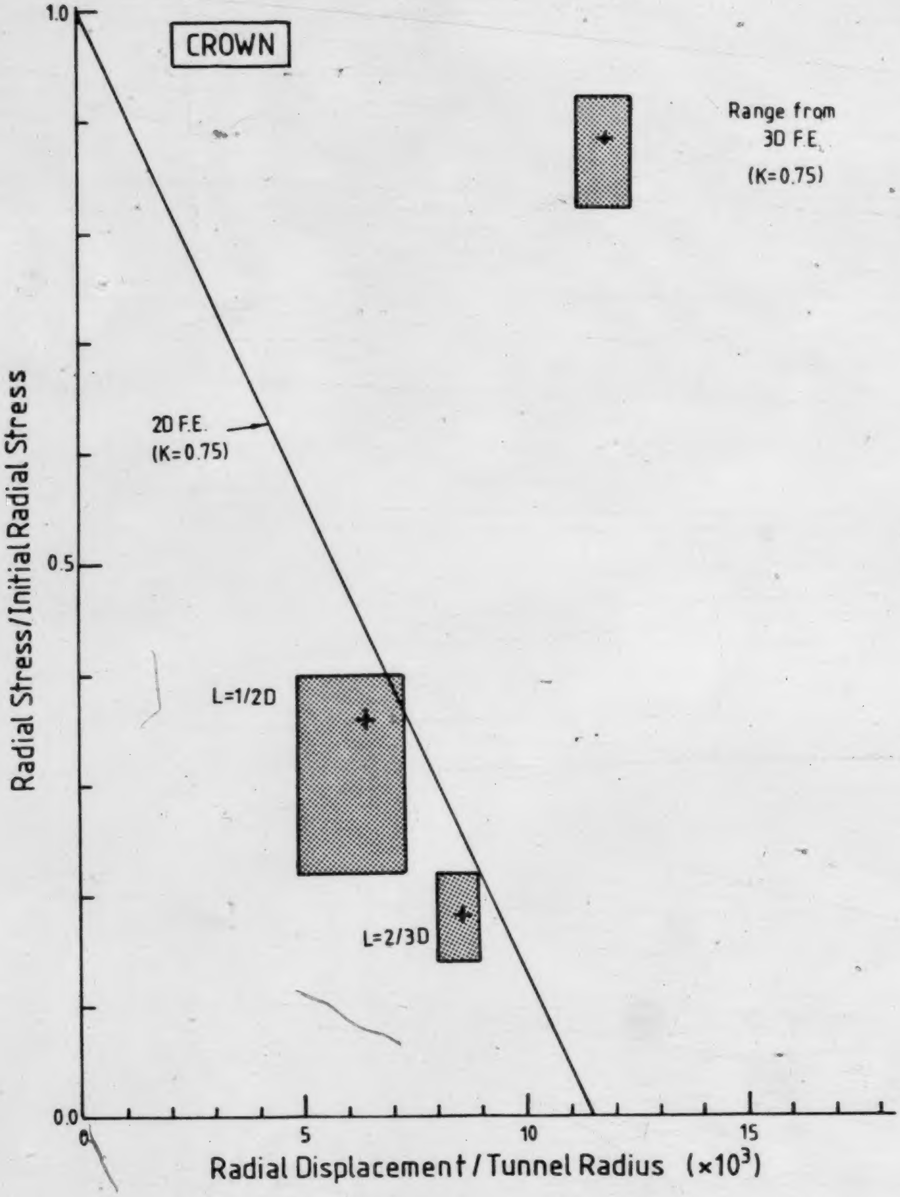


Figure 5.28 Ground–support interaction at crown

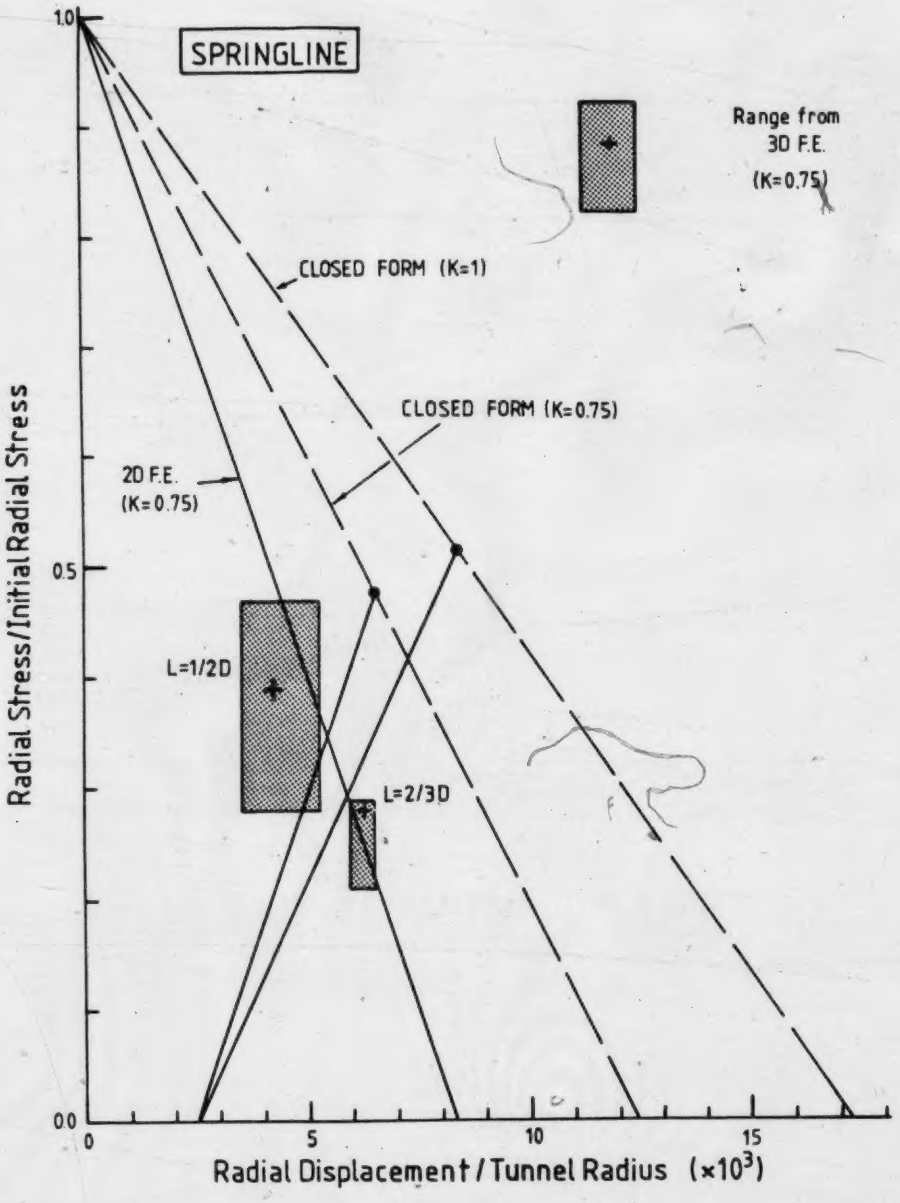


Figure 5.29 Ground–support interaction at springline

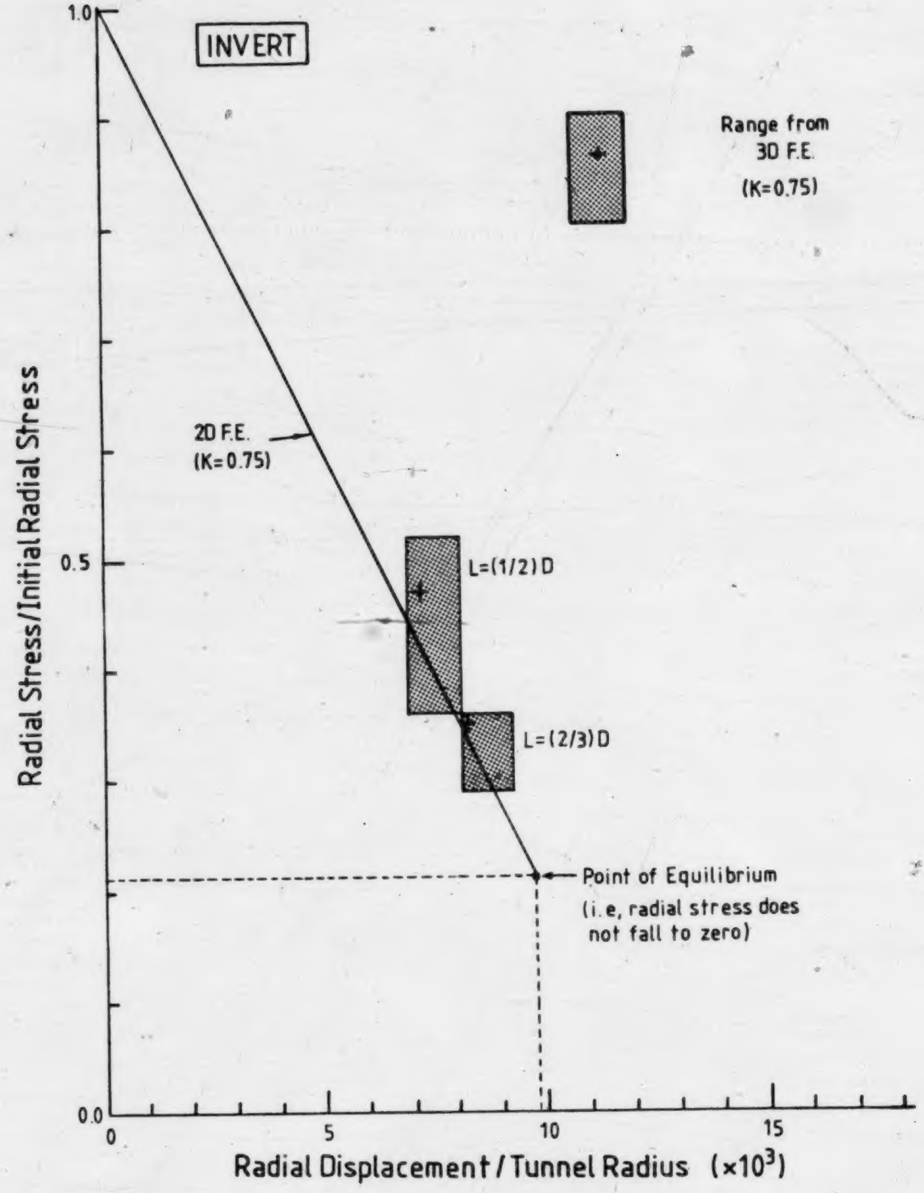


Figure 5.30 Ground-support interaction at invert

above and below the crown is minimized due to the stiffness in the longitudinal direction and to the fact that the excavation was carried out in steps with the support placed immediately after excavation, which minimized the elastic rebound.

Since the springline behaviour is less affected, the following conclusions are mainly based on analysis of Figure 5.29:

1. The previously derived closed form solution provides upper bounds for the problem studied, independent of whether the K used is 0.75 or 1.0. The assumption of K=1, which is normally used in the Convergence Confinement Method, would overestimate the measured loads, but only by a small margin (≃10%) in the case of L=(1/2)D (i.e. where the lining is installed closer to the face). The displacements are also overestimated by using K=1 while with the use of K=0.75 there is a close agreement. It is important to stress that it was assumed that the supports were installed immediately at the face.
2. There is a significant difference between the GRC's determined by the closed form solution and the 2D finite element analysis. This is attributed to the fact that the finite element analysis represents the case of a near surface tunnel, where gravity variation was taken into account. Such is not the case of the closed form solution, which is

rigorously applicable to the deep tunnel case only.

3. Support reaction lines determined by the procedure outlined in Appendix E with the initial displacements (u_s) assumed to be equal to the displacement ahead of the face (u_o), intersected the curves determined by the closed form solution (K=0.75) above the 3D equilibrium points. Hence, in this case this assumption is on the safe side.
4. The equilibrium points given by the intersection of the SRL's with the GRC generated by the 2D finite element analysis do not provide an upper bound for the case where the support was installed at a distance L=(1/2)D. This is attributed to the fact that the support reaction lines were derived from a plane strain solution which does not take into account the in-situ stress variation within the opening's region of influence. It would appear then that support reaction lines should be generated by using results of finite element analyses as well, but this was considered beyond the scope of the present study.

5.4 Concluding Remarks

The material presented in this chapter is essentially composed of two different studies: i.e., the analysis of a case history and a study of the stress-displacement behaviour near the advancing face. The conclusions are therefore presented separately in the following paragraphs.

Observations about the numerical techniques which are important for future studies are reported below.

5.4.1 3D Numerical Analysis

5.4.1.1 General

The only three-dimensional finite element study using ADINA which was known to the Author at the time this research was being initiated was the analysis of the Peachtree Center Station in Atlanta, reported by Azzouz, Schwartz and Einstein (1980). These same authors (Schwartz et al., 1982) presented some of the facts regarding the time required and actual costs of conducting that analysis. Particularly impressive were the spendings – US$18000 for computer time at university research rates, plus four man-months of effort by highly qualified individuals. In a more recent discussion (Schwartz et al., 1983) these authors admitted that the 'time-consuming drudgery of conventional data preparation and output plotting is indeed a waste of valuable engineering talent' and favour the use of 'appropriate graphics software' for this task.

The Author believes that these words reflect his own experience with the three-dimensional analysis reported in this chapter. It is believed that the highest priority for future studies should be placed on the development of efficient pre- and post-processing procedures, possibly making use of 3D graphic devices such as those presented by Han et al. (1983). Computer costs are analysed separately in Appendix D.

5.4.1.2 Numerical Technique

In all three-dimensional analyses carried out in this thesis, fairly continuous stress and displacement curves were found for points ahead of the face (e.g. Figures 5.20 and 5.21). However, for points behind the face, a considerable scatter is evident. This scatter is known to have existed in axisymmetric analyses (e.g. Hutchinson, 1982) and is attributed, in the present study, to the following factors:

1. The size of elements chosen is too large. Areas where the stress and displacement gradients are high require very small elements which in turn increases the cost of the analysis considerably.
2. The length of one round of excavation is not equal to the length of one element in the direction of the tunnel axis. Hutchinson (1982:190) has shown that a smooth displacement distribution curve is obtained if just one slice is excavated per step. He points out however that a smooth curve does not necessarily indicate that the analysis is correct.

In summary, the present analysis resulted in inaccurate values for the displacements and stresses at the tunnel boundary. A technique in which stresses and displacements for three consecutive steps are superimposed was developed and allowed the determination of ranges of equilibrium stresses and displacements around the opening. The analyses of the ABV tunnel presented in Section 5.2 considered displacements of nodes located away from the opening, which were not affected significantly by the innacuracies commented above.

5.4.2 Analysis of ABV Tunnel

The development of displacements ahead of the face was reasonably predicted by the linearly elastic analysis which used Young's moduli values equivalent to the initial tangent moduli in the stress-strain curves[30]. Behind the face, the actual displacements were underestimated, most likely due to the fact that there was actually an increase in the mobilization of ground shear strength which could not be matched because the elastic moduli were not updated during the analysis.

An analysis of the development of displacements at various locations during face advance was carried out in Section 5.2.4. Although merely conceptual due to the absence of a complete set of field measurements, these results might help future interpretation of readings of field instrumentation around NATM tunnels. It was verified that the displacements ahead of the face at the tunnel axis level were largely horizontal, 'flowing' towards the face, whereas the displacements above and below the tunnel tend to be predominantly vertical.

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[30] These moduli were determined by using equation 2 in Figure 4.12, Chapter 4.

5.4.3 Behaviour near the Face

With the intent of characterizing the development of stresses and displacements near the tunnel face, numerical analyses by 3D finite element method were carried out. An analysis of the development of lining loads has shown that these loads occurred during the advance of the face and that smaller loads developed in the support which was placed at a larger distance from the tunnel face.

It was verified that the 3D behaviour resulted in a ground response curve different than that offered by procedures such as the Convergence Confinement Method. The ground response curves shown are of a non-linear nature, even for a material which behaves linearly elastic and time-independent.

Comparison between the 3D results and simplified procedures based on the Convergence-Confinement concepts have shown that these simplified procedures account for the 3D effects. A necessary improvement in these procedures appears to be the inclusion of solutions for situations where the ratio between horizontal and vertical stresses is different than one.

A simplified but apparently more realistic ground response curve was determined by using 2D finite element analysis. Use of numerical techniques might be the way to extend the Convergence Confinement concept to the case of a shallow tunnel.

6. SUMMARY AND CONCLUSIONS

6.1 Introduction

This thesis has examined several aspects related to the geotechnical performance of urban tunnels excavated by the New Austrian Tunnelling Method (NATM). It consisted mainly of four independent studies, namely an overview of the NATM geotechnical performance using data from actual case histories, an analysis of applications of the method to tunnels with large cross-sections, finite element analyses of a case history and a study of the stress-displacement behaviour in areas close to the tunnel face.

In these different sections, attempts were made to relate the development of ground movements to parameters of practical significance. Another contribution to the present work is an assessment of the adequacy of finite element techniques to model shallow tunnels excavated by the NATM.

6.2 Findings from Analysis of Case Histories

Considerable time was spent in collecting data from case histories reported in the literature. Very few of these field cases were documented in sufficient detail, but some improvement in the collection was obtained through personal correspondence with European authors and companies. Most of the data was summarized according to indices commonly found in engineering practice.

The results of this study are listed below:

1. The NATM might not be an appropriate method of soft ground tunnel construction in situations where significant face displacements are anticipated, unless effective ground control measures are implemented. This conclusion has been inferred from the fact that in most published cases the overload factor (OF) was estimated to be below 3, a situation where face displacements are usually not significant. At least one case was reported where the OF value was high (i.e, well above 6) and in this case very large surface displacements were reported. This conclusion is also supported by observations made by Atrott and Sauer (1983) and Duddeck and Erdmann (1982).
2. The NATM regular cross-section tunnels exhibit, in principle, geotechnical performance comparable to shield tunnels. The development of surface settlements can therefore be evaluated by procedures currently used for shield tunnels. It should be pointed out however that some indications, presented in Sections 2.4.3.1 and 2.4.3.3, Chapter 2 suggest that NATM tunnels create displacement fields which are different from those caused by shield tunnels. This is a recommended area for further research.
3. The NATM is a method which allows for an improvement in the geotechnical performance over a certain period of time. This is attributed to the high degree of

flexibility of the method, which easily allows changes in construction features as, for example, the way in which the staged excavation at the face is performed. It is suggested that some startup time may be required until an optimum geotechnical performance can be achieved in a new soil environment using the NATM.

4. Evidence exists, showing that the ground losses in NATM tunnels can be minimized by reducing the invert closure distance, the depth of heading excavation and the invert closure interval. In order to validate these tentative conclusions, instrumentation which allows identification of the displacement mechanisms close to the tunnel is necessary. Further research aimed at gathering such data should be encouraged.

6.3 Large Cross-Section Tunnels

Attention was given to construction of large cross-section soft ground tunnels used in double-track subway tunnels and subway stations. The most efficient way to control ground deformations and maintain face stability in such works is the procedure termed 'staged excavation'.

Typical layouts which have been used in European practice in recent years were reviewed and introduced in a classification system. In geotechnically unfavourable ground there is a tendency to rely in a scheme termed T4, where tunnel advance makes use of side galleries. Selection criteria for the staged excavation scheme were examined but no rigid rules could be established. However, a trend towards a larger number of stages as the cross-sectional area increases is evident from analysis of published cases.

Examination of a few case histories shows that completing the shotcrete invert as close as possible to the tunnel face and as soon as possible after excavation of one round is a major concern, also in these large cross-section applications of the NATM. In some instances, a temporary invert closure within one excavation stage is necessary.

Two–dimensional finite element analyses were carried out to compare relative performance of two staged excavation construction schemes termed T1 and T4. Two extreme conditions believed to represent the Edmonton till were analysed. It was verified that the scheme with side galleries (T4) is superior to the heading and bench scheme (T1) in terms of maximum surface settlements generated. The transverse slope of the settlement trough for scheme T4 is flatter than that for scheme T1. In the stiffer soil, termed 'hard till', the differences in settlement between the two schemes were less significant. It was also verified that excavation of the heading is responsible for a significant amount of the total settlement and alternative schemes with a central support core were proposed.

6.4 Finite Element Studies

6.4.1 Analysis of ABV Case History

Two and three-dimensional analyses were used to back analyse the deformation behaviour around this NATM tunnel. The results suggest that despite the markedly three-dimensional behaviour during the process of excavation, the final transverse displacements can be more economically obtained from two-dimensional plane strain analyses. It was verified however that the calculated settlements were dependent on a parameter termed 'percentage of stress release' (%SR). This parameter represents the amount of stress release with respect to the in-situ condition allowed at the tunnel boundary before lining installation and, in the studies carried out herein, it could not be estimated beforehand. Suggestions for its assessment in future studies were made.

Regarding the three-dimensional analyses, it was verified that even with a coarse mesh, using a linearly elastic isotropic model, aspects of field behaviour could be reproduced. The development ahead of the face was reasonably predicted with the use of moduli equivalent to the initial tangent moduli from conventional triaxial tests. Behind the face however, the actual displacements were underestimated. This is attributed to the increase in ground shear strength mobilization behind the face, which could not be matched because the elastic moduli were not updated during the analysis. It is suggested that future analyses of this same tunnel should be carried out with a non-linear model for the soil.

6.4.2 Behaviour near the Face

In the NATM, the lining is normally placed very near the tunnel face. Three-dimensional analyses to study the development of radial stresses and displacements where the ground and support were assumed to behave linearly elastically and time-independently were carried out. The most important conclusions are as follows:

1. The 3D behaviour resulted in a ground response curve different than that offered by the Convergence Confinement Method. The '3D ground response curve' is of a non-linear nature, even for linear elastic time-independent materials.
2. A comparison between the 3D results and simplified procedures based on the Convergence Confinement concepts has shown that these simplified procedures account for the 3D effects, but overestimate the final stresses and displacements. An improvement was achieved by introducing a $K_0 \neq 1$ condition, compatible with the 3D analyses.
3. A ground response curve generated by a two-dimensional finite element analysis was found to represent the 3D results very closely. It is believed that numerical techniques should be used to extend the Convergence

Confinement Method to the case of near surface tunnels.

6.5 Suggestions for Further Studies

6.5.1 General

Handling the input and output data in the three-dimensional finite element analyses was found to be extremely time-consuming. It is believed that all efforts should be directed to the development of efficient pre- and post-processors, which make use of interactive computer graphics such as those reported by Han et al. (1983).

6.5.2 Analysis of Case Histories

Published case records, although providing quantitative information on the ground movements caused by the NATM, offer only limited insight into the actual mechanisms generating these movements. It is believed that future studies should follow the construction of NATM tunnels, documenting all developments from the site investigation to the instrumentation results. To generate an adequate body of data for identification of the sources of ground loss, intrumentation layouts could be designed following suggestions by Hansmire (1975:237).

6.5.3 Large Cross-Sections

The possibility of excavating large cross-section tunnels appears to be one of the outstanding advantages of the NATM over conventional shield tunnelling. Future research should be directed towards instrumentation of actual field cases and to the development of simplified procedures for the prediction of settlements. Parametric studies using 3D numerical techniques could be used to achieve these objectives.

6.5.4 3D Analyses

Suggestions for future studies involving 3D numerical techniques are:

1. Parametric studies for development of appropriate modelling criteria in 3D analyses (e.g. influence of mesh discretization).
2. A study of the development of support loads and deformations at greater distances from the tunnel face than those analysed in this thesis and involving non-linear soil models.
3. Generation of data against which simplified procedures for use in engineering practice can be checked, following the ideas presented in Section 5.3.7, Chapter 5. Uncertainties regarding ground and support properties would be totally eliminated in such studies, in which the 3D analyses would play the role of a case history or of a laboratory model.

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APPENDIX A - CASE HISTORIES

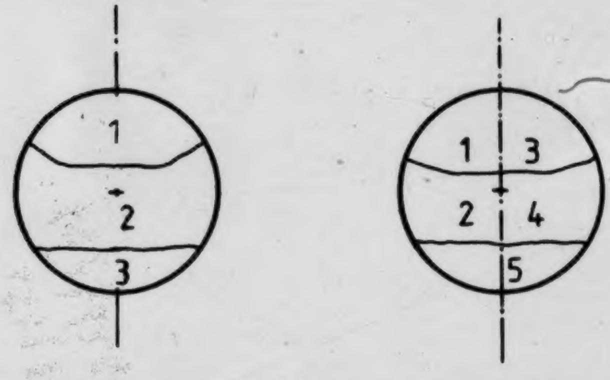
A.1 Introduction

This appendix provides detailed information about seven NATM applications in urban areas. It is hoped that the description of these NATM cases will provide an useful basis from which further investigations can be developed. An effort is made to present typical details of construction and monitoring. It should be pointed out however, that the information collected is intended to be informative only, and considerable care must be exercised in extending past experience to present application. This is especially important because a novel procedure like the NATM is involved.

A.2 Excavation Layouts - Regular Cross-Sections

The focus of this appendix is on tunnels of 'regular' cross-sections. Regular cross-sections are arbitrarily defined as those which do not exceed the dimensions of a conventional single-track subway tunnel (normally 30 to 40m^2 of cross sectional area). This definition is necessary due to the existence of large cross-sections, reviewed in Chapter 3.

Some common excavation layouts for NATM excavated tunnels are depicted in Figure A.1. In the initial applications of the NATM in urban areas a circular cross-section was normally adopted. As shown in Figure A.2,



NUMBERS INDICATE SEQUENCE OF EXCAVATION

Figure A.1 Typical excavation layouts for NATM – regular cross-section

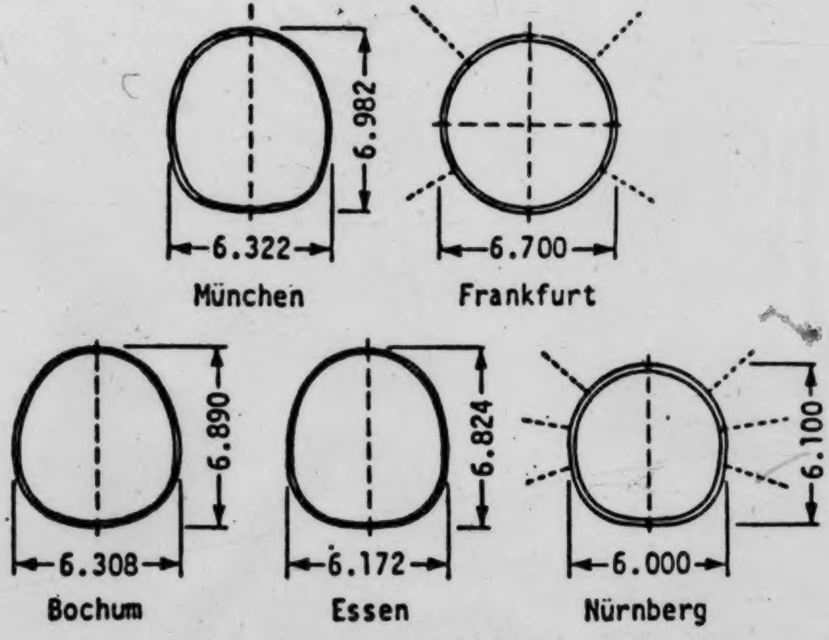

6.982
6.322
München
6.700
Frankfurt
6.890
6.308
Bochum
6.824
6.172
Essen
6.100
6.000
Nürnberg

Figure A.2 Typical cross-section shapes that have been used in West German cities (after Braun, 1982:modified)

some tunnels present departures from this circular shape, the most common being the 'ovaloidal-shaped' type of cross-section.

Although it is apparent that the shape of the cross-section is more dependent on constructional, operational and even architectural details, reference has also been made to its relationship with the in-situ stress field. According to Babendererde (1980a:56) the tunnels in Frankfurt were of circular design because field measurements revealed that in this West German city horizontal stresses did not differ much from the vertical stresses. In other cities like Bochum or Essen, indication existed which showed smaller horizontal stresses, favouring a vertical elliptical-shaped cross section. This is in agreement with theoretical considerations presented in standard textbooks (e.g. Hoek and Brown, 1980:112). Figure A.2 (adapted from Braun, 1982) shows some typical cross-sections of NATM excavated tunnels used in West Germany.

A.3 Case Histories

The NATM was first applied in an urban environment in the city of Frankfurt, the construction of a twin tunnel experimental section beginning in the late sixties. After that, several other tunnels have been completed, mainly in the continental Europe but in other parts of the world as well.

Some of these case histories are thoroughly described in the literature, emphasis normally being placed on construction aspects and field instrumentation results. Unfortunately however, some publications resemble much more advertisement of companies or consultants than proper technical papers. Another problem is the disagreement between different authors which is not uncommon when the number of papers published on a specific tunnel is abundant. Preference in these cases was given to publications of sholarly and research nature.

The number of cases examined was large and a detailed description of each of them would extend this text excessively. Text is provided only for some selected cases, where information considered of significance is reported. Availability of construction details and/or consistent field instrumentation results that could serve as input for future studies were the prime criteria for selecting these cases. Also, some of them exhibit unique construction characteristics that are worthy of notice.

A.3.1 FRANKFURT Baulos 25 - Experimental Section

Being the first application of the NATM in subway tunnels, the Baulos 25[31] twin tunnels have been the object of several papers and are probably the best described in the literature. Special attention is therefore devoted to them in this thesis.

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[31] The german term "Baulos" or "Los" is sometimes translated as 'contract section'. The original terminology will be kept in this thesis.

The experimental section of 50m was excavated and monitored in order to verify the applicability of the NATM to the Frankfurt conditions (Hofmann, 1970). The excavation of the tunnels then went on, the Baulos 25 section hence comprises more than the experimental section alone.

Up to that time, tunneling using shotcrete as primary lining had been exercised only in areas where surface settlements were of little concern. In Frankfurt, the existence of some historical buildings produced the requirement of minimal settlements. The authorities' approval for further excavation using the new method depended strongly on the field instrumentation results of the experimental section. Movements on two empty buildings founded right above the two tubes were monitored. Results of these measurements have been presented by Breth and Chambosse (1975).

Another interesting feature of this case history is that in an adjacent contract section (Baulos 23) two other tunnels were excavated by the shield method. As will be seen in the next items, elsewhere on the shield section geological conditions were less favourable and care should be exercised when making comparisons. Also, the NATM tunnels were excavated simultaneously, as opposed to the shield section.

The main sources for the field instrumentation results presented in following sections are the publication by

Chambosse (1972) and the paper by Stroh and Chambosse (1973).

A.3.1.1 Ground Conditions

The tunnels were excavated in Frankfurt Clay, which according to Breth and Chambosse (1975) is a tertiary clayey marl, occuring beneath a layer of quaternary gravels and sands. The clay is irregularly interbedded with bands of limestone, silts and water bearing sands, being overconsolidated and fissured (Chambosse, 1972:2). Chambosse (op.cit.) further describes the Frankfurt Clay as dark gray to olive gray, with a maximum lime content of 40%. Some geotechnical properties of this clay have been collected in the literature and are presented in Table A.1.

Some horizons of harder material, sometimes refered to as limestone bands, represent another feature of the Frankfurt soils. These stronger layers are of significance because they occasionally had to be blasted (especially in the shield section) and also because they were water bearing. In the NATM section pre-drainage of these layers was provided so that a "dry" excavation was possible (Chambosse, 1972:6).

Table A.1 Typical properties of Frankfurt Clay

				min.	aver.	max.
		BULK MODULUS (KN/m³)	γ	16.5	18.5	20.5
		POROSITY	n	0.30	0.48	0.59
		NAT. WATER CONTENT	ω	0.16	0.32	0.45
		LIQUID LIMIT	ω_L	0.50	0.70	1.10
		PLASTICITY INDEX	I_P	0.32	0.45	0.81
		DEGREE OF SAT.	S	0.80	0.94	1.00
		ACTIVITY INDEX	I_A	0.70	1.00	1.30
STRENGTH		UNIAXIAL COMPRESSIVE STRENGTH (UU) IN MPa	q_u	0.10	0.30	0.60
		FRICTION ANGLE (CD)	φ'	18°	20°	25°
		COHESION IN MPa	c'	0.01	0.02	0.065
DEFORMATION (HYPERBOLIC MODEL)	LOADING	MODULUS NUMBER	K	110	225	240
		" EXPONENT	n	0.50	0.60	0.70
		FAILURE RATIO	R_f	0.70	0.90	0.90
	UNLOAD.-RELOADING	MODULUS NUMBER	K_{UR}	—	650	—
		" EXPONENT	n_{UR}	—	0.60	—

SOURCE: KATZENBACH (1981)

A.3.1.2 Construction Aspects

A plan view of the NATM experimental section is displayed in Figure A.3, which shows the location of the two instrumented sections (MQ I and MQ II). A schematic cross-section showing the geology is displayed in Figure A.4.

The cover (distance from crown to surface) averages 11.5m, both tubes being embedded in Frankfurt clay. The distance between tunnel axes is approximately 12.6m and the excavated diameter 6.4m.

The shotcrete layer was applied in one step (20cm thick), being reinforced with welded wire fabric. Steel ribs were placed at 1.2m intervals. These ribs were fixed to the soil mass by means of 3-4m long anchors (Φ 20mm), which were tamped into the ground (Chambosse, 1972:4). Schulz and Edeling (1973:251) have reported difficulties associated with these anchors, which did not appear to work effectively. Tension tests carried out in the anchors demonstrated that most of them barely contributed to support the tunnels.

As opposed to the shield section (Baulos 23), the NATM tunnels were driven simultaneously (a maximum distance of 6m was allowed between faces (Edeling and Schulz, 1972:359). The reason for this synchronous excavation procedure was to try to minimize settlement distortions at the surface (Edeling and Schulz, 1972:355). The work was carried out by hydraulic

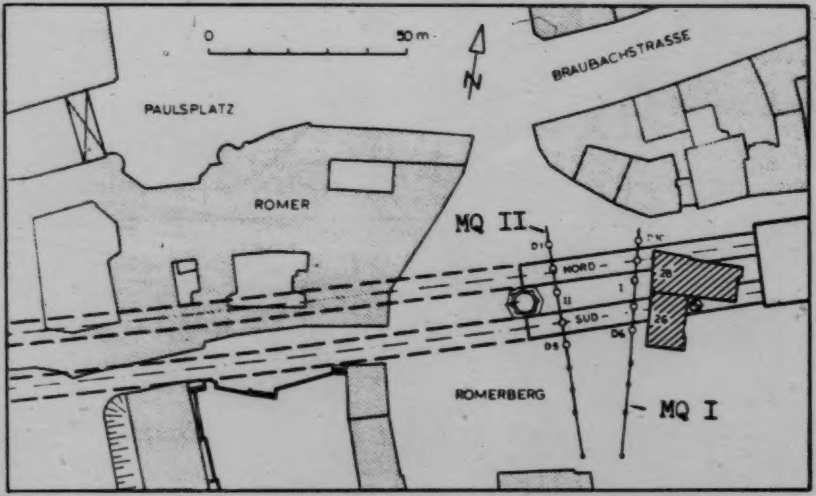
0
50 m
N
BRAUBACHSTRASSE
PAULSPLATZ
ROMER
MQ II
D1
NORD
II
I
SUD
D5
D6
26
28
ROMERBERG
MQ I

Figure A.3 Plan view of Frankfurt Baulos 25 experimental section

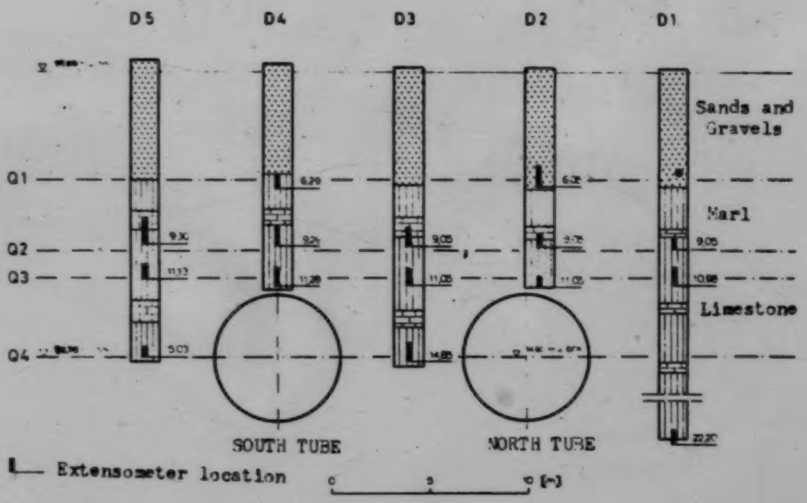
D5
D4
D3
D2
D1
Sands and Gravels
Marl
Limestone
Q1
Q2
Q3
Q4
6.20
9.30
9.25
9.05
11.13
11.20
11.05
10.98
14.85
22.20
SOUTH TUBE
NORTH TUBE
Extensometer location
0
5
10 (m)

Figure A.4 Schematic cross-section of Frankfurt Baulos 25 experimental section(both figures modified from Chambosse, 1972)

excavators (backhoes), the excavated material being removed by wheel loaders.

The majority of limestone bands were removed by the excavator shovel and only occasionally had to be blasted. Edeling and Schulz (1972) report excavation rounds of .75 to 1.20m. The heading precedes the section where the invert is closed by 2.4-5.0m (Chambosse, op.cit.). After each work shift the face was sealed with shotcrete, specifications recommending 5cm for pauses of 1-2 days and 10cm (with eventual mesh reinforcement) for poorer conditions or longer pauses (Edeling and Schulz, 1972:355).

The average rate of excavation for this experimental section was 1.2m/day, increasing to 2.0m/day elsewhere. According to Edeling and Schulz (1972:355), a maximum invert closure interval of 4 days was specified initially[32].

The groundwater was lowered by means of deep wells, installed at 3-5m from the tunnels and which were able to pump an average of 3ℓ/s each. In the NATM experimental section, all the water bearing layers were dried up prior to construction. Other facts about the dewatering measures can be found in the publication by Chambosse (op.cit.).

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[32] It will be seen in subsequent sections that rather shorter invert closure intervals were achieved in other NATM works in Frankfurt.

A.3.1.3 Field Instrumentation Program

The monitoring program was established in order to verify the applicability of the NATM to the Frankfurt conditions (Hofmann, 1970; Edeling and Schulz, 1972). Although a small test gallery had already been excavated and instrumented in 1968 the results were considered insufficient by the city's consultant and a full-scale prototype had to be driven (Schulz and Edeling, 1973:250). A monitoring program which could yield informations about the deformation behaviour of the clay during tunnelling was then designed.

The possibility of obtaining further information from the shield driven section was of great interest as well, because on the potential for comparison due to the similar ground conditions. Five instrumented sections, two for the NATM experimental section and three for the shield section were then implemented. The details of the instrumentation and its results are best described by Chambosse (1972). A plan view of the instrumented sections (MQ I and MQ II) has already been depicted on Figure A.3.

Edeling and Schulz (1972:355) have described the monitoring program, which consisted of (see also Figure A.5):

1. **Pressure measurements** - at both instrumented sections and in both tubes, using the system Glötzl.
   a. Contact pressures (r): 5 contact pressure cells

placed in each instrumented section; area of cell 150x250mm, capable of taking up to 50atm.

b. Concrete stresses (t): 5 pressure cells placed inside the shotcrete ring; area of cell 100x200mm, capable of taking up to 50atm.

2. **Deformation measurements**
   a. Convergence measurements of the ring diameter (k): in 3 cross-sections of the north tube, beginning after invert closure.
   b. Subsurface settlements: measured by means of triple-point extensometers (e).
   c. Fine levelling (n): to monitor surface settlements.
   d. Deformation indicators (d): placed on both sides of the southern tube in both instrumented sections, to measure horizontal deformations[33].

An exhaustive analysis of all instrumentation results falls beyond the scope of this thesis. An extensive collection of data can be found in Chambosse (op.cit.). The measurement results which are pertinent to the present work are summarized in the next sections.

A.3.1.4 Contact Pressures

When interpreting the contact pressure cells readings, one should keep in mind the problems related

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[33] Chambosse (1972:65) also reports horizontal readings of longitudinal displacements towards the face.

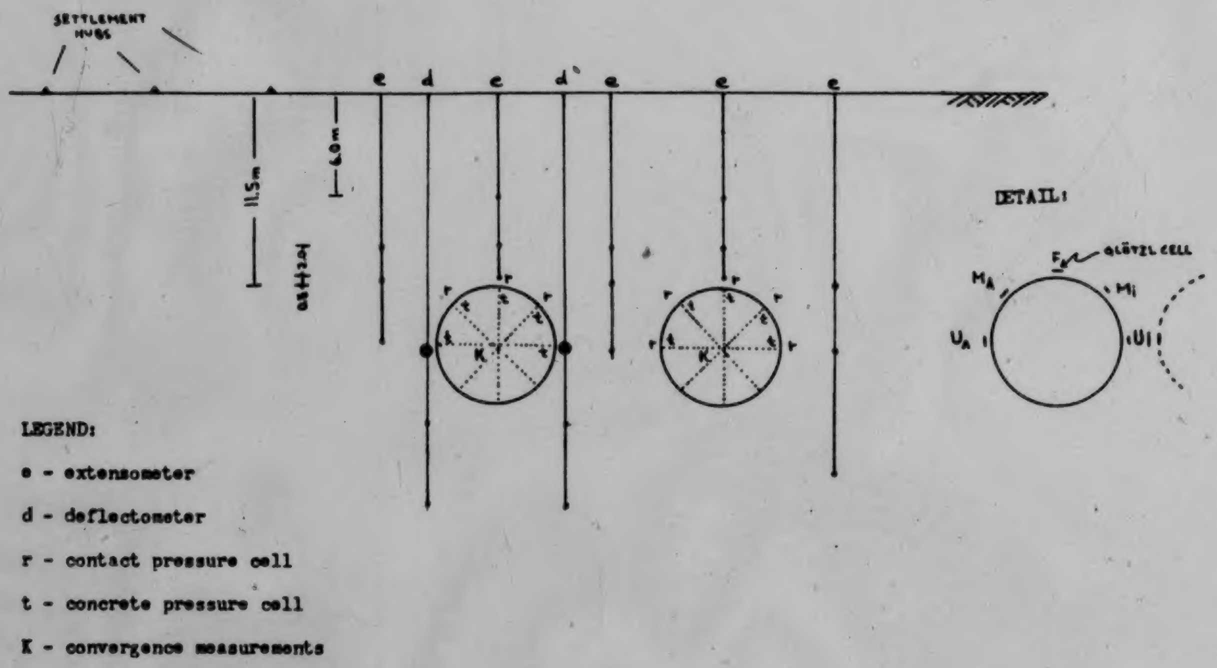

SETTLEMENT HUBS
e
d
e
d'
e
e
e
11.5m
6.0m
offset
r
t
K
DETAIL:
F_A
GLÖTZL CELL
M_A
M_I
U_A
U_I
LEGEND:
e - extensometer
d - deflectometer
r - contact pressure cell
t - concrete pressure cell
K - convergence measurements

Figure A.5 Typical monitoring layout - Frankfurt Baulos 25 experimental section

to Glötzl cells referred to in Chapter 1. The measurements differ considerably from section to section (Chambosse, 1972:21). The average values for three sections presented by Chambosse are 1.07kg/cm² (point F in Figure A.5), 1.60kg/cm² (point Ui), 1.40kg/cm² (point Ua), 1.10kg/cm² (point Mi) and 1.05kg/cm² (point Ma). The minimum value is found at the crown. At the springlines the measurements yielded higher values, the highest value being found at the middle wall.

Chambosse (1972:16) observed that after the shotcrete was placed, the pressure at the crown led to an outward movement at the springlines, activating a pressure of 'passive' nature. However, long term measurements were not reported and a statement about the final stress state is not possible.

An analysis of the stresses measured within the shotcrete ring will not be attempted. Observations reported by Edeling and Schulz (1972:357) suggest that these measurements are not reliable.

A.3.1.5 Horizontal Displacements

Readings of horizontal displacements in the ground mass are somewhat rare to find in the NATM literature. Hence the measurements at the Frankfurt Baulos 25 tunnels have a special significance for research. They have been summarized by Chambosse (op.cit) and are shown on Figure A.6, which has been adapted from that author's work.

Figure A.6a (displacements towards the face) shows that deformations start when the excavated heading is at approximately 2m from the measuring point. Two important observations can be made:

1. movements towards the face occurred even during excavation pauses, being more acentuated the closer the face was to the instrument;
2. the closure of the invert appears to have a noticeable effect on the decrease of these displacements.

Figure A.6b shows the displacements of the springline. Observation of the measurements taken at point L1 show that onset of movement occurs as soon as the heading face excavation reaches the distance of 2m (0.3 diameters) before the test section. Further excavation of the heading is responsible for approximately 45% of the total movement while 50% is due to excavation of the bench. After invert closure displacements tend to stabilize, amounting to only 5% of the total.

Deformations occurring at the springline after the shotcrete was placed were also reported (Chambosse, 1972:66). They were measured by means of a telescopic device. The horizontal diameter increased by an amount $\Delta D = 12mm$ in average. Chambosse (1972:16) reports a correspondent amount of outward displacements (i.e.,

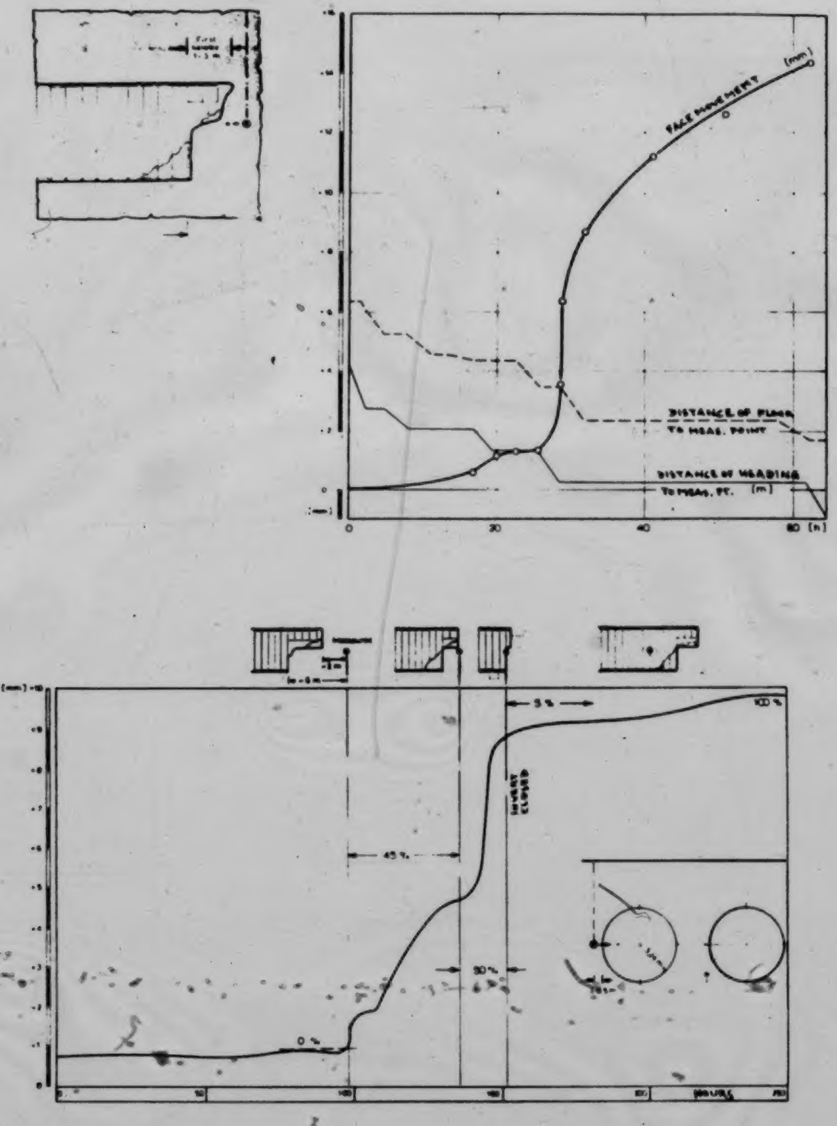


Figure A.6 Horizontal displacements - Frankfurt Baulos 25 experimental section (after Chambosse, 1972:modified)

approximately 6mm on one side) in the soil. The highest gradients ΔD/time took place in the first ten days. The displacements begin to stabilize after about two weeks, reaching equilibrium after six weeks.

Measurement of changes in the vertical lining diameter were also attemped but problems arose due to traffic inside the tunnel. A single measurement of 8mm decrease of vertical diameter is reported by Chambosse.

A.3.1.6 Vertical Displacements

Initial settlements were due to lowering the ground water level. An amount of 35–45mm is reported by Chambosse (1972:7). Chambosse observes that even after the movements due to the tunnel excavation had ceased, a component of 0.5mm/month due to dewatering persisted.

Typical surface and subsurface transverse settlements in section MQ II are shown in Figure A.7. The results for MQ I are very similar.

Maximum transverse distortions at the surface were of the order of 1:300. The settlement trough shape is only slightly altered to a depth of 5.50m, distortion increasing at greater depths. Above the crown of the tunnels, settlements amount to about 60mm (influence of ground water lowering already discounted). The settlement trough in the longitudinal direction started at a distance of about 12m (≈2 diameters) ahead of the face, stabilizing at about 13m (2 diameters) after

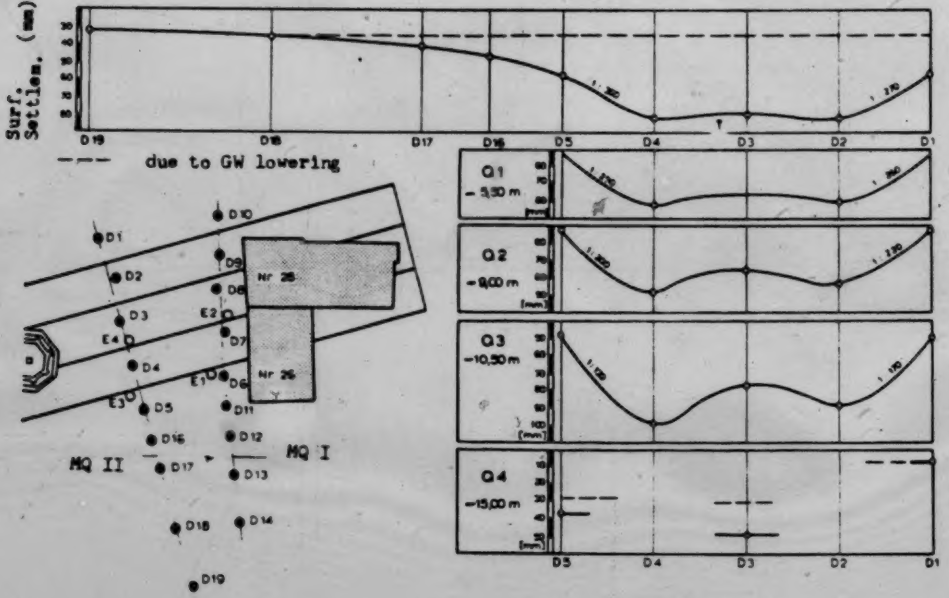


Figure A.7 Surface and subsurface transverse settlements at Frankfurt Baulos 25 experimental section – MQ II (after Chambosse, 1972:modified)

invert closure. The average distortion in the longitudinal direction is of the order of 1:600 (Chambosse, 1972:8).

Assuming that both tubes contribute equally to the total vertical movements, Chambosse (1972:8) estimates the surface settlement contribution of one tunnel to be of the order of 35mm. This value is often reported in the literature but should not be regarded as precise, since the mechanisms involved in the simultaneous excavation of twin tunnels have yet to be clarified.

A.3.1.7 Comparison between NATM and Shield

As already pointed out at the onset of this summary, elsewhere in the shield section (Baulos 23) the geological conditions were less favourable. Furthermore, the shield tunnels were driven one after the other, while in the NATM section they were excavated simultaneously. Nevertheless, these Frankfurt case histories (i.e., Baulos 25 & Baulos 23) embody a unique set of measurements due to the fact that both sections were excavated in similar soil environments and comparisons are often made (e.g. Chambosse, 1972; Schmidt, 1974; Cording and Hansmire, 1975;). However, generalizations about the amount of settlement produced by each method or about the development of this settlement with face advance should be regarded with care.

In order to produce an honest comparison between the two methods, data reported by Chambosse for all instrumented sections of this case study has been summarized and is presented in Table A.2.

In addition, Figure A.8 shows a comparison of surface and crown settlements for both tunnelling methods. It is apparent that the distance of influence ahead of the face (Ld), shown by the surface settlements, is larger in the case of the NATM (≃2 diameters) than in the shield case (≃1.3 diameters). With respect to the percentage of maximum crown settlement occurring ahead of the face, it is seen that the NATM produced the largest percentage (60%). However, data from other NATM tunnels in Frankfurt (Stroh and Chambosse, 1973) show face components which are smaller and closer to the shield case. As seen in Chapter 2, the settlement development is dependent on other factors like the depth of heading excavation, which was larger in the experimental section than in the other tunnels, accounting for the behaviour illustrated in Figure A.8.

A.3.2 Comments

An interesting feature of some of the NATM twin tunnels reported in the literature (e.g. the Los 25 tunnels) is the simultaneous excavation of these tunnels. This is normally not done in shield driven tunnels due to the high costs of implementing the shield. Analyses of cost and geotechnical

METHOD	MEAS. SECTION	max. settlem.		MAX. DISTORTION	EXCAV. DIAM. (m)	SOIL COVER (m)	WATER INFILTRATION	BLASTING OF LIMESTONE
		surf. (mm)	crown (mm)					
NATM	I	35	55	1:350	3.24	11.5	NO	LITTLE
	II	35	50	1:350	3.24	11.5	NO	LITTLE
SHIELD	DOM - South T.	33	50	1:440	3.35	11.8	LITTLE	YES
	DOM - N. Tube	31	45	1:540	3.35	11.8	NO	YES
	FAHRGASSE-S.	69	125	1:140	3.35	9.1	LARGE	YES
	FAHRGASSE-N.	53	70	1:260	3.35	8.8	LARGE	YES
	DOMINIK.G.-S	136	160	1:60	3.35	7.5	LARGE	YES
	DOMINIK.G.-N.	81	130	1:120	3.35	7.7	LARGE	YES

SOURCE: CHAMBOSSE (1972)

Table A.2 Comparison NATM vs. Shield in Frankfurt

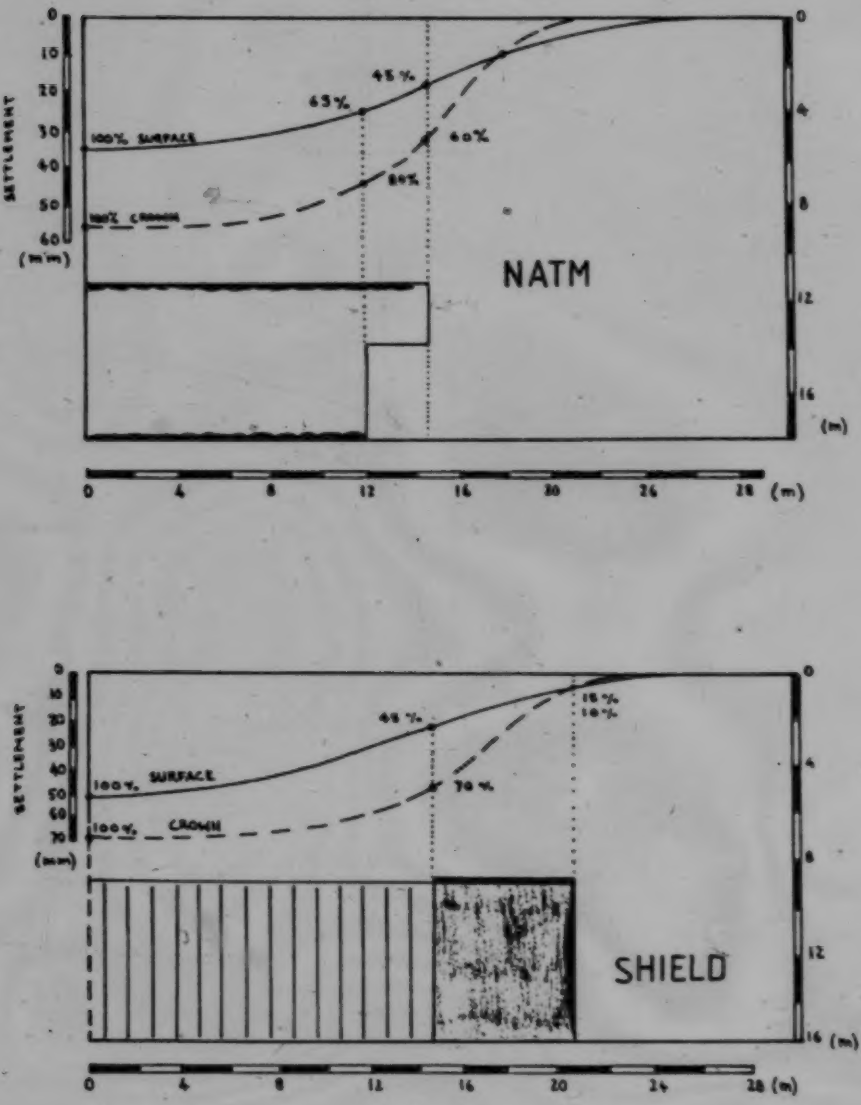


Figure A.8 Settlement development comparison between Shield and NATM in Frankfurt (after Chambosse, 1972:modified)

performance of twin simultaneously and non-simultaneously excavated tunnels would certainly prove useful.

A.3.3 FRANKFURT - Baulos 17 / Baulos 18a

Although there are a number of publications available about these tunnels (Stroh and Chambosse, 1973; Sauer, 1974; Krimmer, 1976; Müller et al., 1977; Blindow et al., 1979), the set of data published is small when compared with the Baulos 25 tunnels. Nevertheless, valuable information is reported and will be summarized herein. As shown in Figure A.9 (adapted from Sauer, 1974), the case history consists of two neighbouring contract sections. Due to route constraint considerations (discussed by Krimmer, op. cit.) the two tubes were not driven simultaneously. The elevation of the floor was gradually changed until the tunnels were situated one above the other.

The soil in question is again the Frankfurt clay described in preceeding sections. Two particular construction details are of importance for the present work and are described in the following paragraphs.

A.3.3.1 Vertical Arrangement of Twin Tunnels

As pointed out above, this solution was sought due to route constraint problems and apparently did not have any relationship with the optimum shape of openings with respect to the in-situ stress field. It is important to observe however that this unique engineering solution

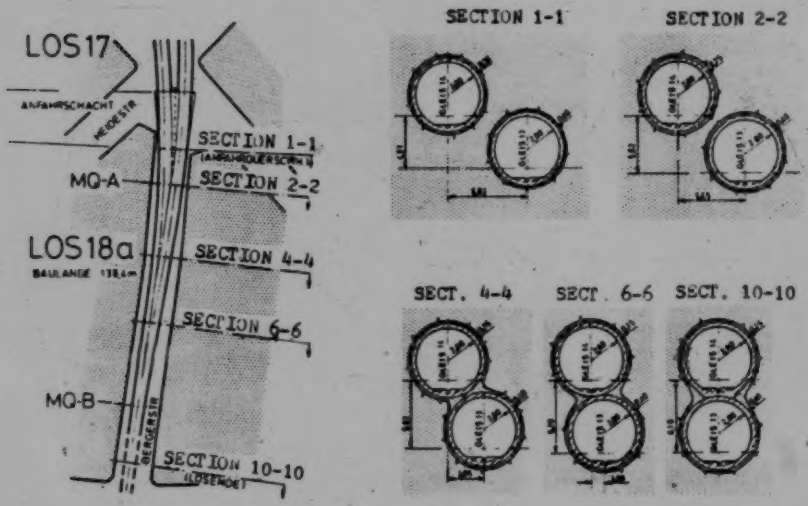
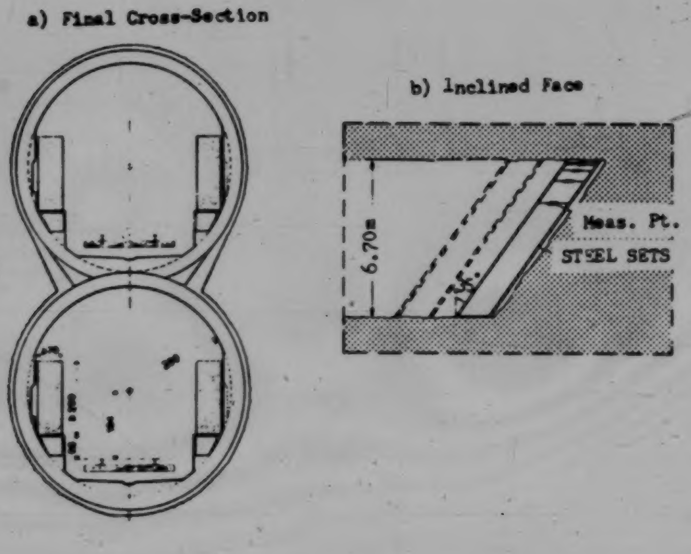

LOS17
ANFAHRSCHACHT
SECTION 1-1
MQ-A
SECTION 2-2
LOS18a
SECTION 4-4
SECTION 6-6
MQ-B
SECTION 10-10
SECTION 1-1
SECTION 2-2
SECT. 4-4
SECT. 6-6
SECT. 10-10

Figure A.9 Plan view and cross-sections – Frankfurt Baulos 17/18a (after Sauer, 1974:modified)


a) Final Cross-Section

b) Inclined Face
6.70m
Meas. Pt.
STEEL SETS

Figure A.10 Construction details – Frankfurt Baulos 17/18a (after Krimmer, 1976:modified)

for tunnelling under a narrow street and low overburden, in a heavily populated area, could hardly be accomplished by conventional shield tunnelling.

The lower tunnel was excavated first, construction of the upper tube beginning only after the secondary lining (35cm of reinforced concrete) was completed (Krimmer, 1976:213). The ground water table, situated at the lower tunnel axis elevation, was lowered prior to construction (Sauer, 1974:14). Figure A.10a shows the final configuration of the tunnels.

A.3.3.2 Inclined Face Excavation

The second innovative feature of this case history was the inclined face and steel sets, a way devised to minimize the heading depth and the invert closure interval, increasing the rate of excavation. According to Müller et al. (1977:1), these factors were found in previous NATM works to be directly related to the magnitude of surface settlements.

The face was excavated in an inclined manner, as depicted in Figure A.10. Small rounds of the order of one meter could be excavated, followed by immediate placement of steel sets and shotcrete. The invert closure interval was lowered to 5 to 8 hours (Sauer, 1974:18) and the surface settlements were kept small (maximum of 30mm). In fact, this arrangement was considered so successfull that the construction of other sections using the same procedure (Baulose 18b1 and 19b1 – Krimmer, 1976; Wagner, 1978) was approved. An alternative would be the cut-and-cover method, which would have caused serious inconvenients in such a densely populated area.

A.3.4 FRANKFURT - S-Bahn Baulos-6

In the preceeding items, sections of the "U-Bahn" system in Frankfurt were described. Rail-guided short-distance passenger traffic in West Germany is divided into networks operated by municipal authorities (tramways if mainly operated on streets and subways or U-Bahn) and those operated by the German Federal Railways. The latter provide suburb connections with large cities and are called 'S-Bahn', which stands for 'rapid transit service' (Martinek, 1979). These tunnels normally have larger diameters due to the wider track spacing which is in turn related to the higher speed of these trains.

The case history described in this item contains a unique set of stress measurements within the ground mass. The main sources of reference are a doctoral dissertation (Sauer, 1976) and two papers where the data are summarized (Sauer and Jonuscheit, 1976; Sauer and Sharma, 1977). The excavated diameter of each tunnel was 7.7m and the ground cover above the crown in the instrumented section reported herein was 14.65m. Both tubes were excavated simultaneously and were fully embedded in Frankfurt clay.

Stress measurements around the tunnels were taken by means of several Glötzl cell clusters. The cells were installed about three months before tunnel construction, carefully packed in bags which were then placed into boreholes. The holes were later refilled with excavated material. Further details on this instrumentation are given by Sauer and Sharma (op. cit.).

Although twenty one sets of readings were published by Sauer and Jonuscheit (op. cit.), only the ones correspondent to points situated above the crown are reported herein (see Figure A.11). The whole monitoring program was aimed mainly at studying the stress changes occuring in the wall between the tubes. It is believed that despite representing the stress changes due to simultaneous excavation of twin tunnels, similar trends would be observed at the crown in the case of a single tunnel.

It is also important to recognize that despite the efforts to obtain reliable stress measurements, these readings do not represent absolute stress values due to unavoidable disturbance of the in-situ stress field caused by drilling the boreholes. However, the readings may serve for a qualitative appreciation of relative stress changes during tunnel excavation.

A.3.4.1 Vertical Pressures

The vertical pressure measurements (Figure A.11a) show an initial stress increase above the initial value

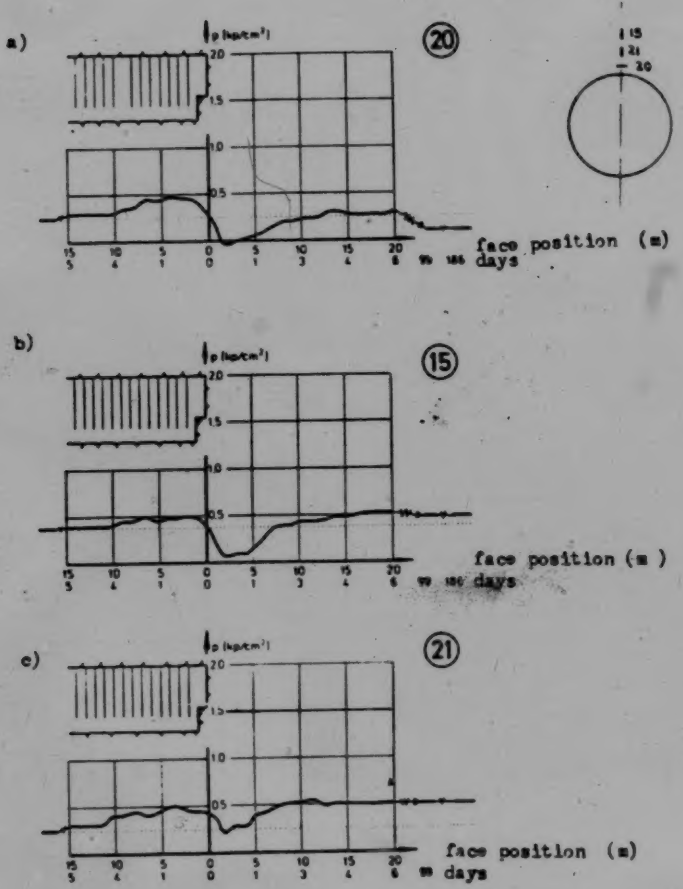


Figure A.11 Stress measurements within the ground mass at crown – Frankfurt S-Bahn Baulos 6 (after Sauer and Jonuscheit, 1976:modified)

as the face approaches the test section. As the tunnel face passes below the instrument the readings fall very sharply down to a minimum which is close to zero. After invert closure (approximately 2.4m behind the face) and further face progress, the readings show a new increase, up to a value which is smaller than the peak ahead of the face. Continued observations have shown that readings have further fallen to a value below the initial value after several months.

A.3.4.2 Tangential Pressures

The tangential pressure readings above crown (points 15 and 21 according to the original notation) are shown in Figures A.11b and A.11c. The trend is similar to that observed for the vertical pressure, but the measurements do not fall to zero as the face passes. Also, the increase to a value above the initial condition is more evident, this second 'peak' being more accentuated than the tangential stress increase ahead of the face.

A.3.4.3 Comments

Stress measurements within the soil mass around tunnels are very difficult to obtain. They always involve the inclusion of a body of different stiffness into the ground, the stress sensor, and this effect, enhanced by installation problems, may yield dubious results. The data reported above is extremely valuable and helps to capture a picture of the complex stress-strain behaviour ahead of the face of a shallow tunnel. It may be re-analysed in connection with results of three-dimensional finite element analyses, in an attempt to characterize the process of load transfer around the face of an advancing tunnel, as shown in Chapter 5.

A.3.5 ESSEN - Baulos 24a

This case history has been described in detail by Hochtief A.G. (1977) and by Steiner et al. (1980). Considerable construction details are reported. Site organization is described herein, with the intent of documenting one case of this aspect of tunneling excavation by the NATM. This subway section of this heavily populated industrial German city comprises the University station (built in an open-cut) and twin single track tunnels of $36m^2$ cross-sectional area. The length of the tunnels is approximately 300m.

A.3.5.1 Ground and Surface Conditions

The tunnels passed under an important university building and also under an embankment of a major railway line. The line run parallel to a road which was also very sensitive to settlements. Ground conditions were a sandy clay (micaceous green sandy marl), with a modulus of elasticity of the order of 300-500kg/cm² (Steiner et al., 1980:319). The ground water table was lowered prior to tunnel construction by means of gravity wells below the invert.

A.3.5.2 Construction Details

The primary lining consisted of steel sets and shotcrete reinforced by welded wire fabric. Support was placed immediately after the excavation of one round of heading, bench and invert. Other aspects which are worthy of notice were the existence of a 'mini-wall beam' ("Vorpfandschienen") tying the steel sets together and the placement of 3m long bolts to tie the steel sets to the ground. Steiner et al. (op. cit.) suggest that the danger of accidentaly tearing off the support with the excavator is a concern and that these bolts greatly reduced this risk.

The average daily rate of advance was 3.5m, the maximum rate being 7.5m/day at the beginning of the tunnels. The rate dropped as excavation progressed due to the use of only one excavator, which alternated between the two tubes (see Figure A.12). Thus, the time to move from one tunnel to the other increased with tunnel length. The excavator (and crew) took 20 minutes to move from one tunnel to the other (for a tunnel length of 200m). Cost considerations then forced the contractor to construct a temporary cross-cut.

The excavated ground was dumped temporarily on the invert. Mucking was also alternated between tunnels, being performed by 3.8m^3 front end loaders. Once the excavator is moved to the other tunnel, the muck is brought to the open cut station.

The support characteristics were directly related to the necessity of minimizing surface settlements. Table A.3a shows that in areas where no buildings existed the round length was larger and the support lighter. Concern with minimization of the invert closure distance is also evident. Table A.3b is a listing of the major equipment and personnel for this tunnel. Other construction and cost details can be found in Steiner et al. (op. cit.).

A.3.6 MUNICH - Baulos 8/1-16

Section 16 of Line 8/1 of the Munich underground system is of interest because a variation of the 'conventional' NATM was successfully attempted. Data have been published by Steiner et al. (1980) and by Nixdorf (1980).

The alignment of these tunnels is S-shaped[34]. The excavation and initial support follows the NATM basic principles but a modification, which has been termed 'Ramp Construction Method' ("Rampenbauweise") has been introduced.

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[34] Possibility of excavating curves with relatively small ground losses is one advantage of the NATM over the shield method.

EQUIPMENT

1	Hydraulic Crawler Excavator Atlas 1602 (150 hp)
1	Front End Loader Gute Hoffnungshutte with 3.8 m³ bucket
2	Shotcrete Pumps GM57 with intermediate silo and accelerator mixer (one for each tunnel)
1	Truck on surface for shotcrete supply + compressor

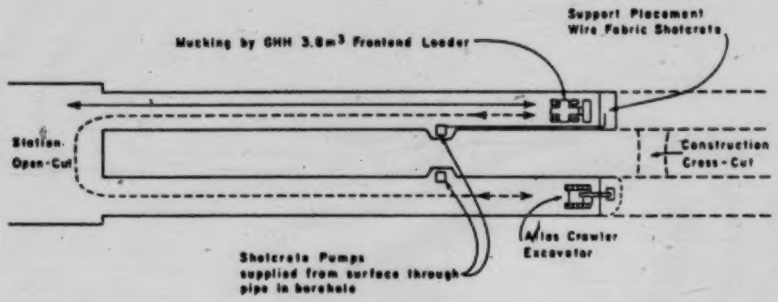
PERSONNEL

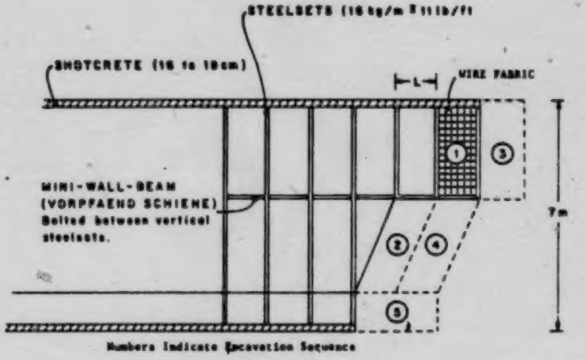
1	Excavator Operator
1	Loader Operator
1	Shotcrete Pump Operator
6	Workmen in tunnels (3 in each)
9	
1	Foreman
10	per shift
2 shifts at 10 hours per day	

INFLUENCE OF SUPPORT AND EXCAVATION PROCEDURE ON DEFORMATIONS

TYPE	ROUND LENGTH = set spacing	STEEL SETS	WIRE FABRIC	SHOTCRETE	RING CLOSURE DISTANCE FROM FACE	SURFACE SETTLEMENTS	CONVERGENCE
No Buildings	1.25 m	16 kg/m in heading & at springline	1 layer	nominal > 16 cm (effective = 20 cm)	6 m	4 cm	1.0-1.5 cm
Under Buildings and Railroad	0.85 m	16 kg/m circumferential	2 layers	nominal > 19 cm (effective = 22 cm)	3.2 m	2 cm	1.0-1.5 cm

Table A.3 Construction and Performance Data - Essen Baulos 24a


Support Placement
Wire Fabric Shotcrete
Mucking by GHH 3.8m³ Frontend Loader
Station
Open-Cut
Construction
Cross-Cut
Atlas Crawler
Excavator
Shotcrete Pumps
supplied from surface through
pipe in borehole


STEELSETS (16 kg/m ≅ 11 lb/ft
SHOTCRETE (16 to 18cm)
L
WIRE FABRIC
MINI-WALL-BEAM
(VORPFAEND SCHIENE)
Bolted between vertical
steelsets.
1
2
3
4
5
7m
Numbers Indicate Excavation Sequence

EXCAVATION AND SUPPORT PROCEDURE AT THE FACE

Figure A.12 Construction procedure - Essen Baulos 24a (after Steiner et al., 1980:modified)

It is described in detail in the following items.

A.3.6.1 Ground Conditions

The Munich soils are usually divided into two main horizons, the quaternary and the tertiary deposits. The superficial quaternary deposits are mainly composed of very pervious sands and gravels. The tertiary soils are composed of sands and marls which are rich in mica. Properties of these materials have been summarized and are presented in Table A.4.

Very unfavourable ground water conditions are frequent in this city. There is normally a free water table in the quaternary sands and gravels and perched water tables are often found in the tertiary layers[35]. Lowering of the water table before construction is normally required in the Munich tunnel works.

A.3.6.2 Ramp Construction Method (after Steiner et al., 1980:modified)

This method has been proposed as an alternative solution by a contractor (Steiner et al., 1980:302), possibly as a compromise which allowed good production rates and good ground control. It incorporates the NATM basic features like shotcrete and steel sets, but the

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[35] Unusual engineering solutions were created to deal with these problems (see Gebhardt, 1980; Steiner et al., 1980 and Krischke and Weber, 1981).

Table A.4 Typical properties of Munich soils

		QUART. GRAVELS	TERT. SANDS	SILTY-CLAYEY MARL
	γ (KN/m^3)	22	21	21
	φ' (°)	30-40	38-40	20-25
	c' (KPa)	0-5	0-5	40-80
	K_o	0.4-0.5	0.5-0.8	0.5-0.8
	ν	0.35	0.35	0.35
E_s	Loading (MPa)	100-140	90-110	100-200
	Unload. (MPa)	200-220	90-180	150-250

SOURCE : KRISCHKE and WEBER (1981)

partial excavation system and the shotcrete placement at the heading present particular characteristics.

Figure A.13 shows a longitudinal section and several cross-sections. The heading is very long when compared to the other case histories already described, preceeding the invert closure by approximately 20 m. It can also be seen that the heading is linked to the invert by a central ramp and benches are left on the side. The bench and invert are excavated simultaneously behind the ramp. Also, the shotcrete support at the heading has extra reinforcements at its base (indicated in Figure A.13), which are sometimes termed 'elephant feet'.

The construction sequence in this particular tunnel is as follows:

1. The heading is excavated by means of a road header (type Westfalia-Dachs). This machine has its own conveyor belt and the muck is directly loaded onto a dump truck which is waiting on the ramp.
2. The heading is supported by steel sets (spaced 0.9 to 1.5m), one layer of welded wire fabric and 15cm of shotcrete. The support of the heading is widened at the springlines in order to provide footings and longitudinal support ('elephant feet').
3. The bench and invert are excavated, followed by invert closure.

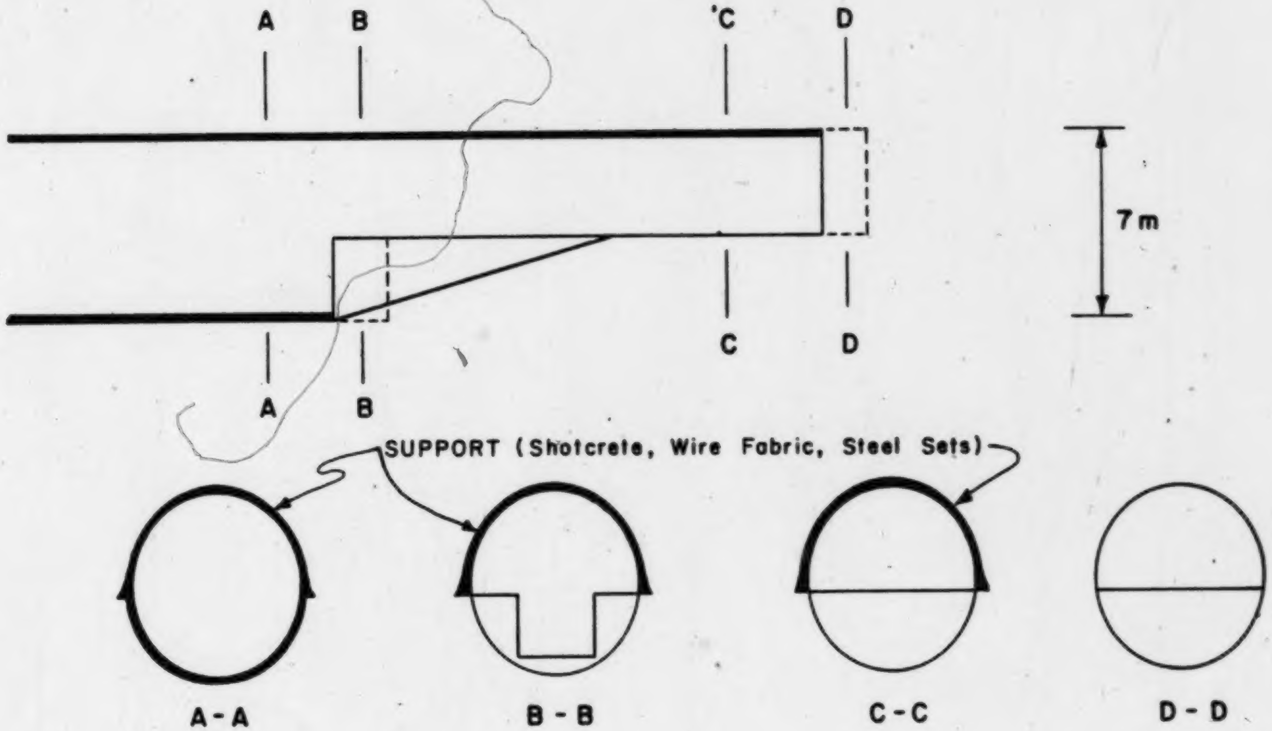
A
B
'C
D
7m
A
B
C
D
SUPPORT (Shotcrete, Wire Fabric, Steel Sets)
A-A
B - B
C-C
D - D

Figure A.13 'Ramp Construction Method'

The use of 'elephant feet' footings plays an important role in minimizing settlements (see Schikora, 1983). It is apparent that the use of this NATM variation allows a much longer invert closure interval with tolerable settlements (see Heilbrunner, 1978).

A.3.7 MUNICH - Baulos 5/9-4.2

The results of field measurements in an experimental section, constructed by the 'Ramp Method' described above were published by Heilbrunner et al. (1982) and are summarized herein. This single tunnel had a cross-sectional area of $38m^2$ and was excavated principally in water bearing tertiary sands[36]. The ground water table in the upper quaternary gravels was lowered prior to construction.

Although the tunnel lining consisted of shotcrete and steel sets, in this 15m long experimental section a special lining was provided. The upper half of the tunnel was lined with steel sets and steel plates, the lower half with the same steel sets and shotcrete (see Figure A.14a). The reason for this was the installation of instruments, as reported in the following paragraph.

A.3.7.1 Field Instrumentation Results

In order to obtain the stress distribution across the steel ribs, strain gauges[37] were attached to them. These measurements were then used to determine both normal forces and bending moments in the lining, presumably by a procedure similar to that outlined by El-Nahhas (1980:156).

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[36] Typical properties of the Munich soils were reported in the preceeding section.
[37] The original text refers to 'deformation measuring

Figure A.14b and A.14c show the evolution of the calculated normal forces with time in the two measuring rings (MQ I and MQ II). Heilbrunner et al. (op.cit.) recognize four different phases in the normal forces ("N"-forces) development (Figure A.14b):

Phase 1: Within the first 36 hours after placement of the steel sets, the normal forces raise rapidly to about 85kN.

Phase 2: The normal force values increase mildly to values between 110 and 120kN until invert closure.

Phase 3: Bench and ramp excavation and invert closure, which took approximately four days, cause an increase of about 30kN in normal forces.

Phase 4: The values tend to stabilize right after invert closure at a value equivalent to about 50% of the full overburden pressure, staying constant for at least 210 days (time during which measurements were taken).

Figure A.15a shows the results for Phase 1 in more detail. It is noticeable that the evolution of normal forces (and consequently of the load at crown) is highly

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[37](cont'd)devices' which were installed to web and flanges of the steel ribs.

dependent on the construction procedure. A face advance provokes an increase in the loads and excavation interruptions cause a stabilization. This shows that the development of lining loads, in this specific case, is highly dependent on the face advance. No time dependency is evident.

Figure A.15b shows the evolution of surface settlements with face advance. Results of another measuring section, with same geology but where steel sets and shotcrete were placed, are depicted as well. It can be seen that surface settlements ahead of the face are of the order of 35% of the maximum value. It is also apparent that the use of a mixed lining system (shotcrete in the lower half of the tunnel and steel plates in the upper half) yields settlements of magnitude comparable to those of the 'full shotcrete' section.

A.3.8 São Paulo - ABV Tunnel

This hand-mined tunnel forms part of a connection between two water treatment plants in the southern area of São Paulo, Brazil. Construction started in 1978. Extensive construction details were published by Lotito and Carneiro da Cunha (1978), field instrumentation results being given in the papers by Negro and Eisenstein (1981) and Simondi et al. (1982). Additional information was obtained from reports on design and field instrumentation.

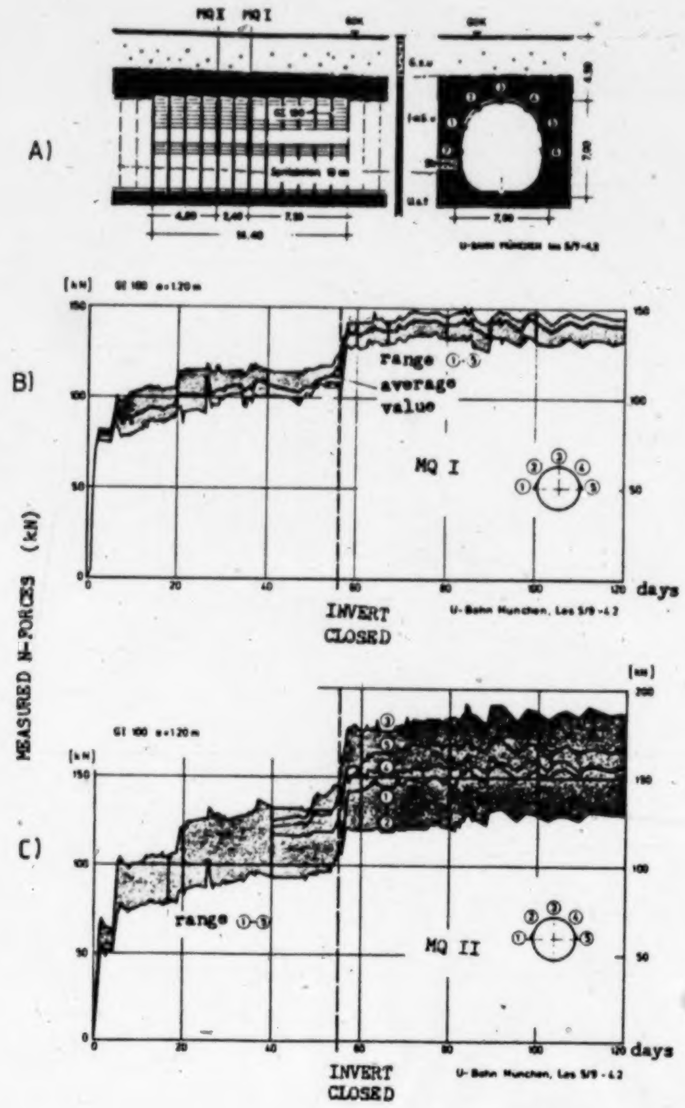


Figure A.14 Development of normal forces in the lining system - Munich Baulos 5/9-4.2 (after Heilbrunner et al., 1982:modified)

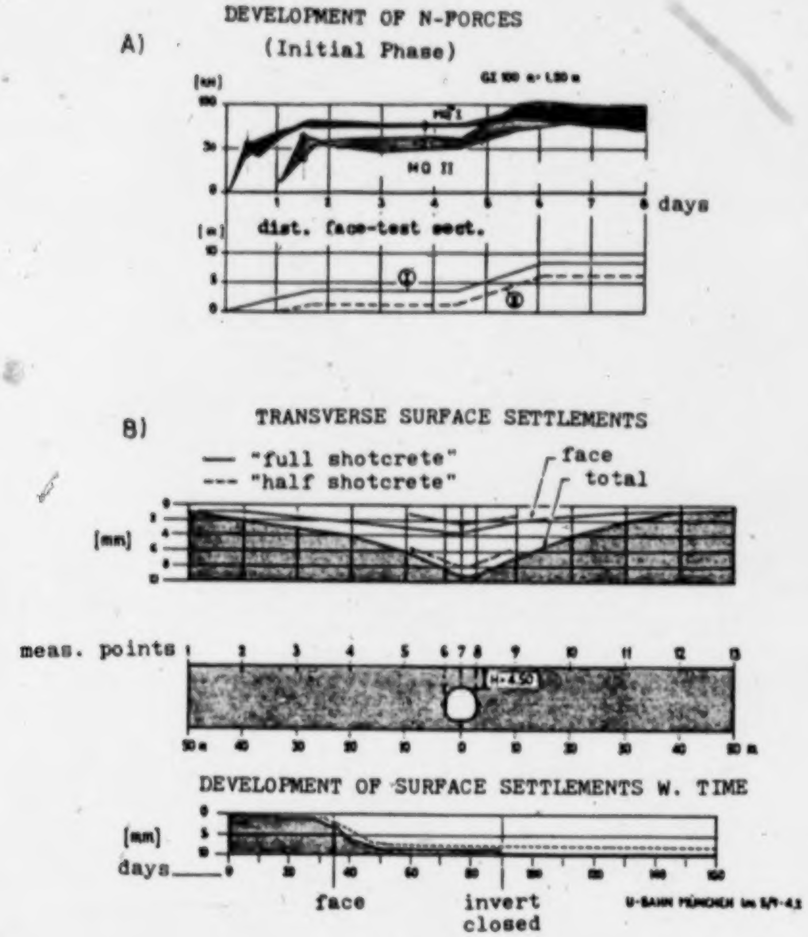


Figure A.15 Field measurements at Munich Baulos 5/9-4.2 (after Heilbrunner et al., 1982:modified)

Steel tubes (Φ 98") were later placed into the tunnels. In addition to the NATM, two other tunnelling methods were used, in order to compare costs and construction performance. Similar geological conditions were found along the 450m connection. The soil cover above the NATM sections averages 6.3m.

Figure A.16 presents a plan view of the site. The instrumented sections referred to in this thesis are also depicted. The cut-and-cover, liner plates and horseshoe steel ribs section had been used before in similar jobs. The NATM was a new procedure whose efficiency had to be evaluated.

A.3.8.1 Ground Conditions

The area in which the tunnels were excavated is situated in the southern extremity of the São Paulo sedimentary basin and is characterized by a smooth topography, elevations oscillating between 760 and 770 (meters). A detailed regional study of these soils has been presented by Pichler (1948). The bedrock existent below the basin is formed mostly by rocks of Pre-Cambrian age, which are covered by tertiary sediments.

One of this deposits has been termed 'variegated soil', due to the variety of colors that it presents. It is a very heterogeneous material, whose composition varies from sand to clay. Overlying these soils there is

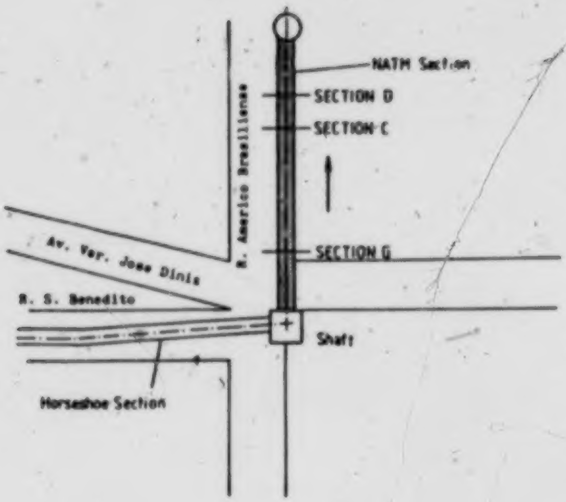
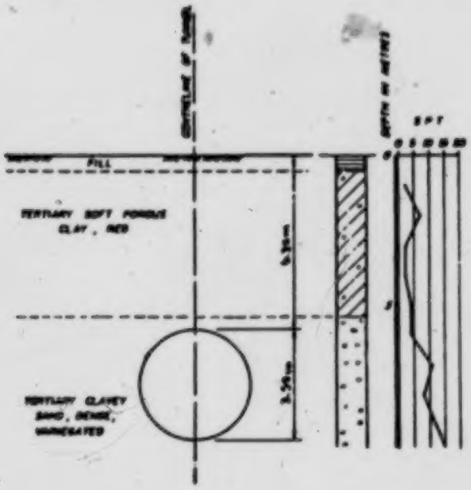

NATM Section
SECTION D
SECTION C
SECTION G
R. Americo Brasiliense
Av. Ver. Jose Dinis
R. S. Benedito
Shaft
Horseshoe Section

Figure A.16 Site of São Paulo ABV tunnels (after Simondi et al., 1982:modified)


CENTERLINE OF TUNNEL
DEPTH IN METERS
SPT
FILL
TERTIARY SOFT POROUS CLAY, RED
TERTIARY CLAYEY SAND, DENSE, VARIEGATED
0
5

C

Figure A.17 Typical geotechnical profile at ABV site (after Negro and Eisenstein, 1981:modified)

a layer of tertiary, soft, reddish clay, which presents a porous macrostructure. Due to these characteristics, reference is frequently made to the São Paulo 'red porous clay'.

Figure A.17 depicts a typical profile at the area being dealt with in the present work. The profile is relatively uniform along the entire length of the tunnel. Also shown is a typical Standard Penetration Test (SPT) profile. Various types of laboratory and field (pressuremeter) tests were carried out on the local soils. The results for the variegated soil agree with those of Group II in the classification system outlined by Sousa Pinto and Massad (1972). This classification system is of interest to the present work because it includes hyperbolic parameters (according to Kondner and Zelasko, 1963), which are used in the finite element analyses in Chapters 4 and 5 of this thesis. A summary of properties of the variegated soil is presented in Table A.5.

The lateritic porous red clay is a very compressible material whose deformation properties present a strong heterogeneity. Results of Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT), carried out in an area adjacent to the ABV project indicate noticeable differences between boreholes spaced only 6m (Massad et al.,1981). A summary of properties obtained during the construction of the

ABV tunnels is presented in Table A.5.

The in-situ stress field, more specifically the K_0[38] value, is of vital importance in design and in numerical analysis. A value of $K_0=0.5$ was adopted in the design. For the present study, another approach for K_0 determination was used. It is based on the study published by Mayne and Kulhawy (1982). Adopting the values shown in Table A.1 (OCR=2.0 and $\phi'=29°$ for the variegated soil) and entering the expressions by Mayne and Kulhawy (op.cit.), one obtains $K_0=0.72$. Work by Massad (1981) yields a value of 0.78 for the porous clay, which is in agreement with that determined above. For the analyses carried out in this thesis, K_0 values ranging from 0.43 to 0.75 were considered.

A.3.8.2 Construction Aspects

The availability of the 'as-built' reports provided a number of valuable execution details. For the sake of brevity, most of them will be summarized in the form of figures. Particular additional informations related to the numerical back-analyses are presented in Chapters 4 and 5.

Ground water was found below the invert of the tunnels and no dewatering was necessary. The hand mined

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[38] The coefficient of earth pressure at rest K_0 is defined in terms of effective stresses. In tunnel engineering this rigorousness is frequently relaxed and K_0 sometimes expresses simply the ratio between the in-situ horizontal and vertical stresses. This thesis considers the proper terminology as long as adequate data are available.

Table A.5 Typical properties of ABV site soils

VARIEGATED SOIL:

PHYS. INDICES	BULK DENSITY γ (kN/m^3)	18.2
	LIQUID LIMIT w_L (%)	48
	PLASTIC LIMIT w_P (%)	19
	PLAST. INDEX I_P (%)	29
	NAT. WAT. CONTENT w (%)	20
STRENGHT	UNCONF. COMPRESSIVE STRENGHT q_u (kPa)	80
	FRICTION ANGLE φ (CU)	21°
	COHESION INTERCEPT c (MPa)	0.04
DEFORMATION (HYPERBOLIC)	MODULUS NUMBER K	330
	MODULUS EXPONENT n	0.23
	FAILURE RATIO R_f	1.0 (assumed)

$\varphi' = 29°$

OCR = 2.0

POROUS CLAY:

PHYS. INDICES	γ (kN/m^3)	15.2
	w_L (%)	65
	w_P (%)	33
	I_P (%)	32
	w (%)	34
STRENGHT	UNCONFINED COMPRESSIVE STRENGHT q_u (kPa)	100
DEFORMATION MODULI	OEDOMETER	
	Primary Loading (kg/cm^2)	70
	Unloading (1.5 → 0.5 kg/cm^2)	230
	Reloading (0.5 → 1.5 kg/cm^2)	220

excavation was carried out with the aid of jack hammers. Plate A.1 depicts an aspect of the heading excavation. The muck was removed from the excavation front and placed on a conveyor belt[31]. For an advance of 1m in the heading, 1.2 hours were required. The same advance in the bench and floor required 1.6 hours (Simondi et al., 1982).

As excavation progressed, the crew acquaintance with the general nature and properties of the ground, associated with the results of field instrumentation showed that the design could be altered as construction went on. The heading advance was increased from 1.0 to 1.2–1.5m. The bench advances (rounds of 1.90–2.20m) were carried out after two heading excavation rounds. Figure A.18 shows the constructive scheme in more detail.

The shotcrete, which was used as a definitive support, was normally placed after the wire mesh had been attached to the excavation walls, In the heading area, an initial protective layer was provided, in some instances even before the mesh had been placed. The shotcrete at the invert and springlines was placed in one step only, having a final thickness of about 10cm (see Plates A.2, A.3, and A.4). Finally, the second layer was sprayed in the heading area. The maximum invert closure distance (i.e., distance between the face and completed invert) was approximately 4m, the

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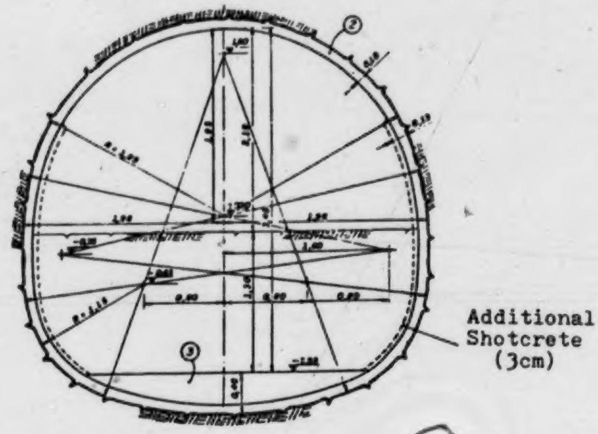
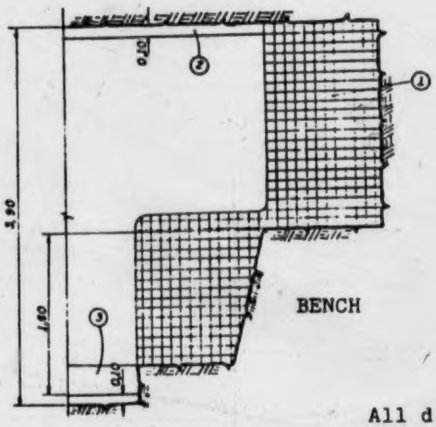
[31] According the Simondi et al. (1982), this system proved to be ineficient for distances greater than 50m.

CROSS-SECTION


②
③
Additional
Shotcrete
(3cm)

① Wire Mesh (Q159 - 2.521g/cm^2 - 10x10cm)
② Shotcrete (thickness 10cm)
③ Floor filled w. reflected shotcrete

LONGITUDINAL SECTION


0.20
②
①
3.90
1.80
③
0.10
BENCH

All dimensions in meters

Figure A.18 Construction scheme - São Paulo ABV tunnel (after THEMAG ENGENHARIA, 1979:modified)

correspondent time interval approximately 4–5 hours.

Light weight steel sets (I4" spaced 0.8-1.2m) were used only in some unfavourable areas, where water infiltration from old utilities occurred. This corresponded to about 10% of the tunnel length. The wire mesh (openings 10x10cm) did not play a structural role in the design and its use was aimed at improving the adhesion of the shotcrete to the soil and at minimizing the occurence of tension cracks. Simondi et al. (op.cit.) report a perfect condition of the shotcrete, in a site inspection carried out 40 months after tunnel completion.

An overall construction rate of 1.5 to 2.9m/day is reported by Simondi et al. (op.cit.). During weekend pauses the face was sealed with a shotcrete layer.

A.3.8.3 Field Instrumentation Program

Details have already been presented by Negro and Eisenstein (1981) and will only be briefly reviewed herein. The location of the three instrumented sections in the NATM tunnels has already been shown in Figure A.16. All sections consisted of three rows of five surface settlement monuments. Magnetic multi–point extensometers were provided for sections G and C only. The extensometer points were located above the tunnel crown, 'within' the tunnel (these were cut when face passed the control section) and below the invert. The

Plate A.1 ABV tunnel - heading excavation

Plate A.2 ABV tunnel - side wall immediately after bench excavation (photos courtesy of Mr. A. Negro Jr.)

Plate A.3 ABV tunnel - area near the excavation face

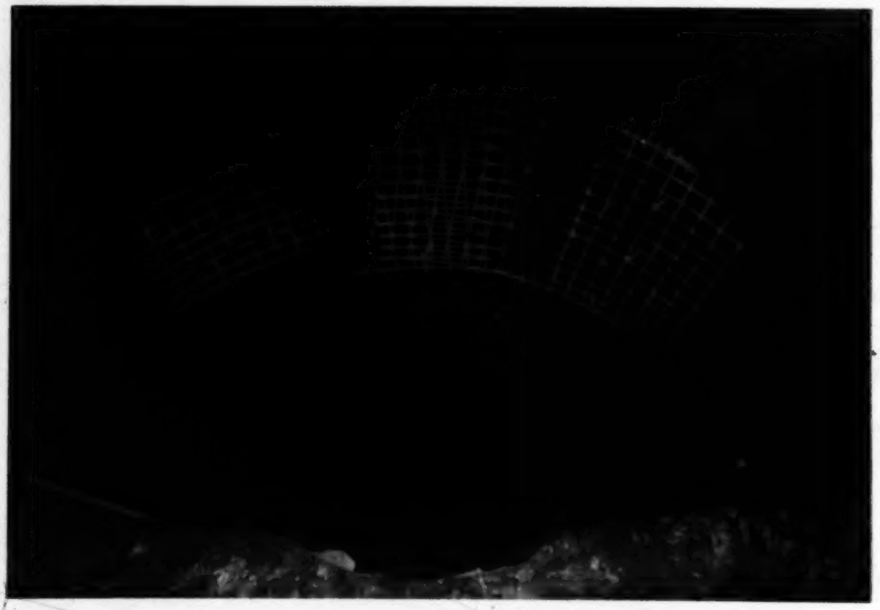


Plate A.4 ABV tunnel - detail of mesh placement at the heading (photos courtesy of Mr. A. Negro Jr.)

extensometers were similar to type C magnet holders as described by Marsland and Quarterman (1974).

Lining convergence measurements were carried out with the aid of a tape extensometer (see El-Nahhas, 1980). These measurements were accompanied by a level survey of settlement pins installed in the shotcrete, at the crown. Precision of ±0.25mm was achieved by using a Wild W3 level and an INVAR levelling rod.

The majority of instrumentation results are analysed in Chapters 4 and 5, in connection with the numerical analyses. Some of them are also summarized in Figures A.19 and A.20.

A.3.8.4 Displacements Ahead of the Face

Although no inclinometers were installed and thus no information about horizontal displacements is available, important information can be derived from the vertical displacements measured. Figure A.21 shows schematically the vertical displacements as a function of the face position (Section C). It can be seen that measurable displacements start to occur at distances of approximately 4 to 6m ahead of the advancing heading. This corresponds to 1.0 to 1.5 diameters.

It can also be noticed that the movement at the invert (point C7) is of an upward nature, reaching a final value corresponding to approximately 60% of the settlement above crown. It is also apparent that these displacements below the invert start after the

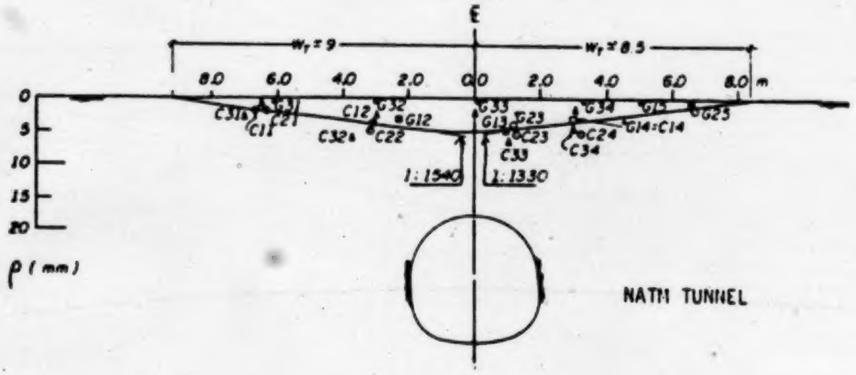


Figure A.19 Surface settlements – São Paulo ABV tunnel

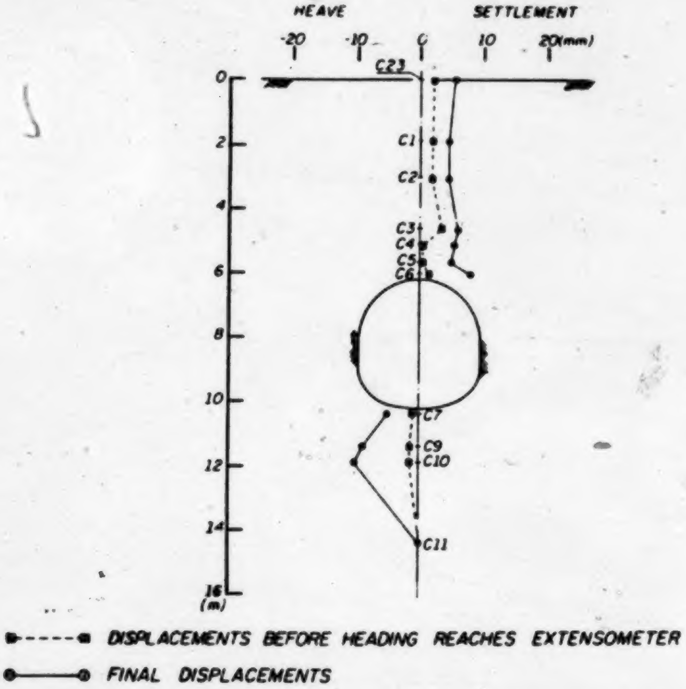


Figure A.20 Subsurface settlements – São Paulo ABV tunnel (both figures modified after Negro and Eisenstein, 1981)

displacements above the crown, but reach their final values faster. This is due to the removal of the bench and is of importance in specifications of time spacing of extensometer measurements.

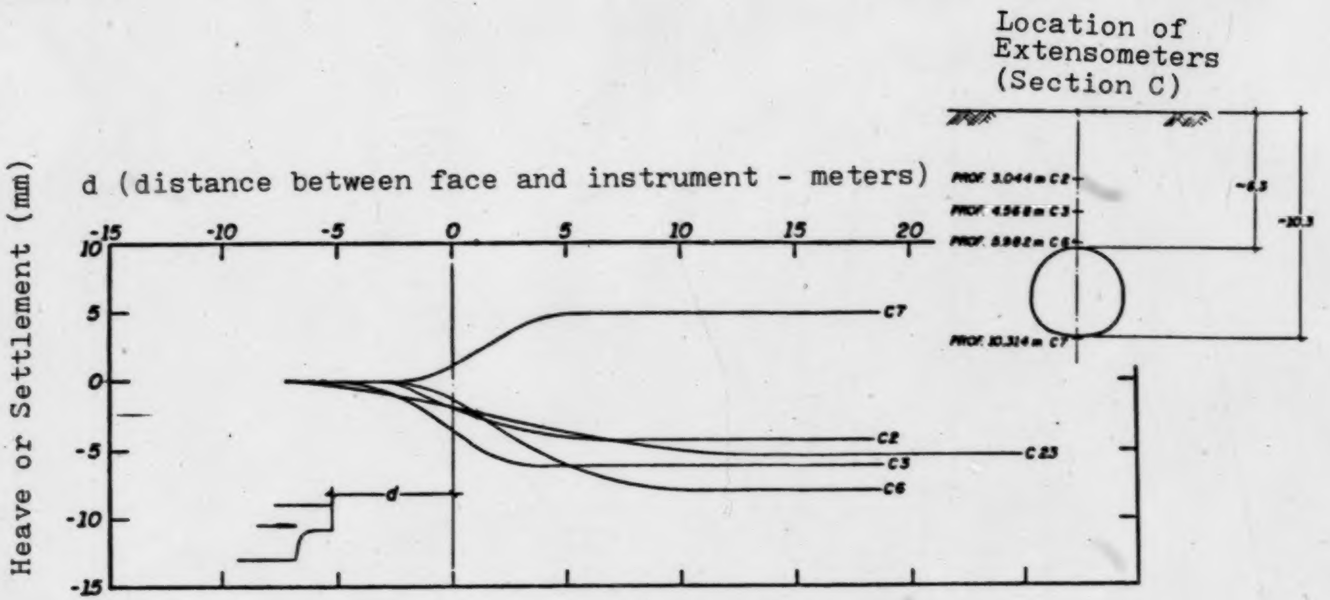


Figure A.21 Vertical displacements with face position - São Paulo ABV tunnel (after THEMAG ENGENHARIA, 1979:modified)

APPENDIX B - TABLES

This appendix contains tables which summarize field data from several case histories. These data were analysed in Chapter 2.

Table B.1 Case histories investigated

No.	CASE	GROUND TYPE	MAIN REFERENCES	REMARKS
1	ALTMUHL Test Gallery	soft sandstone	Wittke et al.(1974)	FM
2	BOCHUM Baulos A2	marls	Jagsch et al.(1974) Hofmann(1976)	CD,DD,FM large cross-section
3	BOCHUM Baulos A3-A5	marl+sandy silt +gravel	Laue et al.(1978) Blindow et al.(1979)	CD,FM includes subway station
4	BOCHUM Baulos B1	marl+silty sand +gravel	Golser(1979)	CD,FM includes subway station
5	BOCHUM Baulos B3	marl+silt+ +gravel	Hunt(1978) Wittke/Gell(1980)	FM
6	BOCHUM Unterfahrung Hbf.	marls	Schulze(1981)	CD,FM large cross section
7	BUTTERBERG	sandy silty gravels	Meister/Wallner(1977)	CD,FM large cross-section
8	ENGELBERG Test Sect. 1	marl	Kuhnhenn/Lorscheider (1979)	CD,FM
9	ESSEN Baulos 18	silty marl +coal(invert)	Stadt Essen(1981)	CD includes subway station
10	ESSEN Baulos 24a	sandy clay	Steiner et al.(1980) Hochtief(1977)	CD,DD,FM twin tunnels-simultaneous excavation
11	FRANKFURT Dominikaner-gasse gallery	Frankfurt clay (tertiary clayey marl)	Edeling/Schulz(1972) Schulz/Edeling(1973)	CD,FM
12	FRANKFURT Baulos 25	Frankfurt clay	Chambosse(1972) Edeling/Schulz(1972) Stroh/Chambosse(1973)	CD,DD,FM twin tunnels-simultaneous excavation
13	FRANKFURT Baulos 17	Frankfurt clay	Stroh/Chambosse(1973) Muller et al.(1977)	CD,DD,FM twin tunnels
14	FRANKFURT Baulos 18a	Frankfurt clay	Sauer(1974) Krimmer(1976) Blindow et al.(1979)	CD,DD,FM twin tunnels
15	FRANKFURT S-Bahn Los 6	Frankfurt clay	Sauer/Jonuscheit(1976) Sauer/Sharma(1977) Sauer(1976)	CD,DD,FM twin tunnels-simultaneous excavation
16	FRANKFURT S-Bahn Los 5.2	Frankfurt clay	Rottenfusser(1974)	CD,FM multiple excavation
17	FRANKFURT Baulos U-27	Frankfurt clay	Schultz(1976)	CD,FM twin tunnels-simultaneous excavation
18	FRANKFURT Baulos 18b1-19b1	Frankfurt clay	Wagner(1978)	CD twin tunnels
19	FRANKFURT Baulos U-82	Frankfurt clay	Babendererde(1980a,b) Brem(1981)	CD, ,FM subway station

CD - Construction details available in the literature
DD - Detailed description included in this thesis
FM - Field measurements available in the literature

Table B.2 Case histories investigated (contd.)

No.	CASE	GROUND TYPE	MAIN REFERENCES	REMARKS
20	MUNICH Baulos 5/9-4.2	sand+marl	Heilbrunner et al. (1982)	CD,DD,FM 'Ramp Method'
21	MUNICH Baulos 6-West 2	gravel	Schikora(1982) Schikora(1983)	CD,FM twin tunnels
22	MUNICH Baulos 8/1-5.1	marl+sand +gravel	Heilbrunner(1978)	CD,FM twin tunnels
23	MUNICH Baulos 8/1-9	marl+sand +gravel	Laabmayr/Weber(1978) Laabmayr/Pacher(1978)	CD,FM large cross-section
24	MUNICH Baulos 8/1-18.2	marl+sand +gravel	Laabmayr/Weber(1978) Laabmayr/Pacher(1978)	CD,FM
25	MUNICH Baulos 8/1-8.1	marl+sand +gravel	Golser et al.(1977) Nixdorf(1980)	CD,FM twin tunnels-simultaneous excavation
26	MUNICH Baulos 8/1-14	marl+sand	Nixdorf(1980)	CD
27	MUNICH Baulos 8/1-16	marl+sand +gravel	Steiner et al.(1978) Nixdorf(1980)	CD,DD 'Ramp Method'
28	MUNICH (non-ident.)	marl+sand +gravel	Krischke/Weber(1981)	CD,DD,FM large cross-section
29	NURNBERG Hasenbuck	soft sandstone	Blindow/Wagner(1976) Hofmann(1976)	CD,FM large cross-section
30	NURNBERG Lorenzkirche	soft sandstone	Bauernfeind et al. (1978)	CD,FM twin, large cross-section
31	NURNBERG Schweinau	soft sandstone	Gartung/Bauernfeind (1983)	CD,FM twin, large cross-section
32	PARIS Grigny	tertiary marls	Egger(1975)	CD,FM large cross-section
33	REGENSBURG Pfaffenstein	variable-tert. sands/clays+ sandst.+limest.	Hereth(1979) Blindow/Wagner(1978)	CD large cross-section
34	SCHWAIKHEIM	marls+ limestone horizons	Rabcewicz/Sattler (1965) Muller(1978)	CD railway tunnel first application of the NATM
35	SAO PAULO Alto da Boa Vista	tert. clayey sand	Negro/Eisenstein(1981) Simondi et al.(1982)	CD,DD,FM Finite Element Analyses presented in Chaps 4/5
36	SAO PAULO Metro North Extension	silty sands clays	Cruz et al.(1982) Celestino et al.(1982)	CD, ,FM twin tunnels
37	SAO PAULO Almeida Lima	fine clayey sand	Teixeira(1982)	CD
38	SAO PAULO Cassandoca	clayey sand	Teixeira(1982)	CD,FM

CD - Construction details available in the literature
DD - Detailed description included in this thesis
FM - Field measurements available in the literature

Table B.3 Overload Factor vs. Loss of Ground

Case No.	Pv (kPa)	Cu (kPa)	OF	Loss of Ground Vt (m3/m-%Vtn)	REMARKS
4	141	50	2.8	0.40-0.63	Section I
12I	272	150	1.8	0.45-1.40	
12II	302.5	150	2.0	0.32-0.92	
12III	284	150	1.9	0.26-0.74	
13IV	191.5	150	1.3	0.22-0.63	
13V	247	150	1.6	0.17-0.48	
14	295.0	150	2.0	0.14-0.39	
23	271	250	1.1	0.21-0.28	
24	546	250	2.2	0.08-0.23	
25	424	250	1.7	0.11-0.30	
35C	136	50	2.7	0.04-0.28	
35G	136	50	2.7	0.02-0.16	
36A	230[1]	20?	11.5	1.05-3.70	1st of twin

Case No.	C(m)	D(m)	y(m)	Ss(mm)	St(mm)	Sf(mm)	Sm(mm)	Sn(mm)	Vf(1) (m3/m-%)	Vm(2) (m3/m-%)	Vn(3) (m3/m-%)	Vt=1+2+3 (m3/m-%)	Vs (m3/m-%)	REMARKS
2	12.2	9	0.5	17.6	20.0	-	-	-	-	-	-	0.20-0.31	0.32-0.51	MO I
4	3.6	9	0.5	41.0	41.0	-	-	-	-	-	-	0.40-0.63	0.42-0.67	Section I
7	14.6	11	2.1	10.0	23.0	-	-	-	-	-	-	0.34-0.36	0.22-0.23	
8	26.0	6.4	0.5	28.0	56.0	-	-	-	-	-	-	0.44-1.37	0.75-2.35	
12	11.5	6.4	0.5	48.0	61.0	38.0	13.0	10.0	0.28-0.87	0.10-0.30	0.07-0.23	0.45-1.40	0.75-2.32	MO I
12	13.0	6.7	0.5	37.0	42.0	14.0	15.0	13.0	0.11-0.31	0.12-0.33	0.10-0.28	0.32-0.92	0.63-1.79	MO II
12	12.5	6.7	0.5	28.0	34.0	13.0	9.0	12.0	0.10-0.28	0.07-0.20	0.09-0.26	0.26-0.74	0.46-1.29	MO III
13	7.0	6.7	0.5	23.0	29.0	6.0	14.0	9.0	0.05-0.13	0.11-0.31	0.07-0.20	0.22-0.63	0.27-0.77	MO IV
13	10.0	6.7	0.5	19.0	22.0	4.0	9.0	9.0	0.03-0.09	0.07-0.20	0.07-0.20	0.17-0.48	0.28-0.79	MO V
14	12.6	6.7	0.7	11.0	17.0	-	-	-	-	-	-	0.14-0.39	0.18-0.52	Gleis 13-MO A
21	6.0	6.6	0.5	4.1	7.5	-	-	-	-	-	-	0.06-0.17	0.04-0.13	1st tube-MO IV
23	8.0	9.8	0.4	20.0	40.0	-	-	-	-	-	-	0.21-0.28	0.31-0.40	
24	22.0	6.8	0.6	3.5	10.5	4.0	5.0	1.5	0.03-0.09	0.04-0.10	0.01-0.03	0.08-0.23	0.09-0.24	
25	17.0	7	0.7	7.0	13.0	4.0	7.5	1.5	0.04-0.10	0.07-0.18	0.01-0.04	0.11-0.30	0.14-0.39	MO II
35	6.3	3.9	0.2	5.50	8.00	1.5	4.0	2.5	0.01-0.05	0.02-0.14	0.01-0.09	0.04-0.28	0.05-0.39	Section C
35	6.7	3.9	0.5	2.50	4.00	1.5	2.5	0.0	0.01-0.06	0.01-0.10	-	0.02-0.16	0.03-0.21	Section G

Table B.4 Components of Settlement and Loss of Ground

Table B.5 Trough Width Parameter vs. Tunnel Depth

Case No.	z (m)	a (m)	i (m)	i/a	z/2a	REMARKS
4	7	3.50	3	0.86	1.00	Section II
12I	14.7	3.20	4.8	1.50	2.30	Cording/Hansmire(1975)
13	14.1	3.35	7.2	2.20	2.10	
14	16.0	3.35	7.2	2.20	2.50	Gleis 13-MQ A
28	14.1	5.30	10-12	1.4-2.3	1.33	Krischke/Weber(1981:121)
28	17.0	5.00	15.4	2.91	1.70	"
28	18.2	5.00	22.3	4.46	1.82	"
31	7.3	4.05	2.7	0.67	0.91	1st tunnel only
32	17.0	5.00	4-10	0.8-2.0	1.70	
35	8.25	1.95	2.93	1.50	1.58	Average Sections C/G
38	4.5	1.90	1.7-2.5	0.9-1.3	1.16	

Table B.6 Quality of Construction

Case No.	D (m)	Ss (mm)	Vtn (m3/m)	Vs (m3/m)	REMARKS
2	9	5	63.6	0.32	
4	7	8	38.5	0.42	Section II
7	11.0	12	95.0	0.22	
8	6.4	28	32.2	0.75	
12I	6.4	48	32.2	0.74	
12II	6.7	37	35.3	0.63	
12III	6.7	28	35.3	0.46	
13IV	6.7	23	35.3	0.27	
13V	6.7	19	35.3	0.28	
14	6.7	11	35.3	0.18	
21	6.6	4.1	34.2	0.137	
23	9.8	20	75.4	0.31	
24	6.8	3.5	36.3	0.09	
35	3.9	5.5	11.9	0.05	Section C

Vs estimated assuming trough properties as in Fig. 2.1 (Chapter 2)

Table B.7 Invert Closure Distance / Invert Closure Interval

Case No.	Ss(mm)	D(m)	E(MPa)	&(kN/m3)	norm. Ss (x1000)	L(m)	T(hrs.)	REMARKS
2	16	9	30-50	20	0.3-0.5	2.5	-	
4	8	7	5-10	20	0.04-0.09	1.0	-	Section II
4	70	7	4-10	20	0.33-0.78	2.0	-	Section IV
7	12	11	60-80	20	0.30-0.40	8.0	-	
10	20	6.6	30-50	20	0.69-1.15	3.2	-	
10	40	6.6	30-50	20	1.38-2.30	6.0	-	
13	19	6.7	20-50	18.5	0.46-1.14	4	8	
13	23	6.7	20-50	18.5	0.55-1.38	5	12	
14	11	6.7	20-50	18.5	0.26-0.66	-	5-8	
17	31	6.8	20-50	18.5	0.72-1.81	3	18	
18	11	6.7	20-50	18.5	0.26-0.56	-	12-24	
35	5.5	3.9	30-50	18.0	0.60-1.00	4	7	Section C

APPENDIX C - DESIGN CONCEPTS

C.1 Introduction

This appendix summarizes information related to design of linings in urban tunnels. The main sources are from German literature and emphasis is placed on features that are peculiar of NATM tunnels.

Results of a recent survey by the International Tunnelling Association - ITA (1982:219) shows that temporary and permanent NATM supports are often designed by the same method but with different load assumptions. It should also be pointed out that there is a currently a trend to use the shotcrete as a permanent support (Eckschmidt and Celestino, 1982). In these cases, the support is augmented in two or more stages at increasing distances from the face and the usual distinction between temporary and final support tends to disappear.

C.2 Lining Design Directives

In engineering practice, it is sometimes appropriate to have guidelines which provide a general design framework. The following guidelines have been adapted from the paper by Duddeck (1980). They presumably reflect in at least a general manner the philosophy of soft ground tunnel design existent in West Germany, from where most of the case histories examined in this thesis come from. Its presentation herein is not aimed at establishing 'rules' for NATM design but to provide some ideas that could perhaps be incorporated elsewhere.

The design directives have been elaborated by a working group of the German Society for Soil Mechanics and Foundation Engineering. In a former edition, these recommendations were restricted to the design of shield driven tunnels but were later expanded in order to include the design of tunnels excavated according to the NATM. A complete review of the recommendations will not be attempted herein. Of relevance to the present work are the proposed calculation models, as well as the various load assumptions (i.e. earth and water pressures, temperature effects, etc.). The original notation is maintained herein.

C.2.1 General

It is important to highlight some points that describe the general character of the German recommendations:

1. The working group made a special effort to keep the recommendations as simple as possible.
2. Text, formulae and figures refer to circular tunnels but an extension to other types of cross-section should be possible.
3. Although it is recommended that the 3D effects in the face area should be investigated 'when the construction method makes it necessary', in all reccomended

structural models a two-dimensional situation 'far from the tunnel face' is assumed.

4. The use of linear elastic models for the ground and for the lining is allowed.
5. Unavoidable uncertainties associated with tunnelling are normally large and should be reflected in any numerical statement.
6. Sensitivity studies which encompass the upper and lower values of soil parameters are highly encouraged.
7. The use of tunnelling instrumentation is also reccomended.

C.2.2 Parameters influencing the Design

The following points and parameters are known to affect the calculation results and are listed in order of importance (Duddeck, 1980:351). The list might be used as a guide to sensitivity studies:

Parameters of decisive influence:

1. Earth pressures - specially the coefficient of lateral earth pressure and an eventual consideration of a percentage of stress release.
2. Water pressure.
3. Choice of static system.
4. Choice of soil parameters.

Parameters of considerable influence:

5. Stresses due to lining assemblage, curve driving and

nearby tunnels.

6. Late influences such as traffic vibrations and time-dependent properties of the ground.
7. Temperature variations in the tunnel longitudinal direction, specially temperature decrease.
8. Compression pressures due to the shield (not applicable to NATM).
9. Injection pressures.

Parameters of little influence

10. Self-weight of the tunnel lining.
11. Installation and traffic loads inside the tunnel.
12. Air pressure.
13. Temperature differences between inside and outside the tunnel.

C.2.3 Design Loads

The following list has been adapted from Duddeck (op. cit.):

1. Earth Pressures

   The state of stress existent in the ground before the tunnel is driven is called the 'primary state of stress' and is normally calculated by conventional assumptions (i.e., γz and $K_o \gamma z$). An assumption of K_o=0.5 is allowed. Water pressures should be added whenever water is present, as well as surcharges due to foundations and/or traffic (see points 2 and 3).

The soil pressures on the permanent support are always taken as being these 'primary pressures', based on the assumption that the ground pressures will return to the in-situ conditions after a long period of time (Duddeck and Erdmann, 1982:83). Allowance for a percentage of stress release is not explicitly recommended but apparently allowed for the design of the temporary support (Duddeck, 1980:355) [40].

It is also important to comment that the German recommendations point the necessity of verifying the possible state of failure above the crown when such stress release is considered. If the ground shear strength is fully mobilized, the 'silo pressure' of the overburden should be adopted as crown surcharge for tunnels with $h/d \geq 3$, where h is the depth to crown and d is the tunnel diameter.

2. Water Pressure

The water pressure may be computed as the water column above the tunnel times the water density (i.e., $\gamma_0 z$). In general, the highest water level (WL) results in the maximum thrust (compression loads) while the lowest WL yields the maximum bending moment in the lining (Duddeck, 1980:351).

3. Foundation and traffic loads

---

[40] Duddeck and Erdmann (1982:91) suggest that for tunnels excavated by the NATM a reduction in loads might be justified. This statement is grounded on the assumption that the NATM will require a soil with good mechanical properties and supports one of the conclusions of Chapter 2 of this thesis.

The self-weight and live loads[41] from structures and the loads due to the action of vehicles may be taken into account as distributed loads, applied respectively at the foundation's base elevation or at the road's elevation. The increase in the earth pressures due to these loads should be propagated with an angle of 60° with respect to the horizontal, as shown in Figure C.1. This assumption applies to surcharges acting on one side only, as well as to loads from isolated deep foundations. In general, even when surcharges act on one side only, the lateral pressure can be assumed symmetric. The introduction of impact allowance factors in the structural design is generally not required for covers larger than 3.50m.

4. Other loads

Liner self-weight

The self weight of the liner can in general, be neglected. It may eventually be taken into account by introducing a crown surcharge[42].

Installation and traffic loads inside the tunnel

These loads are normally of little significance, but may be considered as complementary distributed loads.

Temperature effects

---

[41] Muir Wood (quoted in ITA (1982:191)) points that live loads are not usually serious.

[42] It appears that for a thin shotcrete shell the self-weight may be neglected without any major consequence.

Calculation assumptions for uniform and non-uniform temperature changes have to be determined for each particular case, depending on operational conditions and on the construction stage. The highest air temperature occurs mostly during lining placement[43].

Air pressure

When compressed air is used an impervious lining should be assumed for calculation purposes.

C.2.4 Shallow and Deep tunnels

The German recommendations make distinctions between shallow and deep tunnels, as shown in Figure C.2. The adopted distinction seems rather arbitrary, but might reflect local experience. Duddeck (1980:352) explain the use of these models in the following manner:

1. **Shallow tunnels (h<2d)**, where, h=soil cover above crown and d=tunnel diameter):

   In this case, models should be chosen which consider the ground mass collaboration only in the areas where the lining deforms towards the ground[44]. It is normally sufficient to neglect embedding at crown only, the invert being left with support.

2. **Tunnels with larger cover (h>3d)**:

---

[43] Muir Wood (in ITA, 1982:191) points that 'temperature and humidity changes may need to be controlled during construction' of shotcrete tunnels.

[44] It should be noted that depending on the K_0 value the crown will move outwards or inwards. This is a recognized oversimplification in the german recommendations that has been criticized for example by Katzenbach (1981).

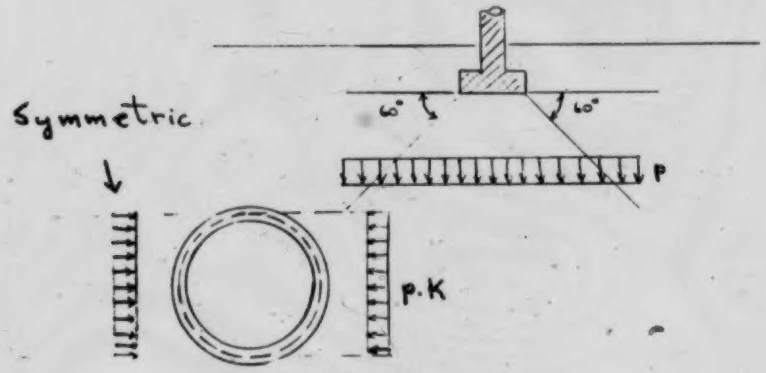

Symmetric
60°
60°
P
p.K

Figure C.1 Proposed distribution of foundation loads

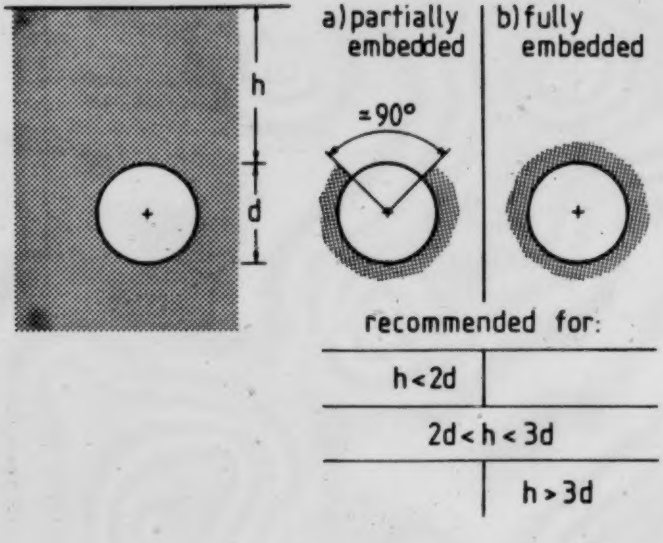

h
d
a) partially embedded
b) fully embedded
≈90°
recommended for:
h<2d
2d<h<3d
h>3d

Figure C.2 Distinction between shallow and deep tunnels

The static calculations may now be performed with a model which takes embedding at the crown into account[45].

3. **Intermediate zone (2d<h<3d):**

This 'transition' is apparently left as a matter of engineering judgement. Either model may be adopted depending on the ground conditions. Duddeck (op.cit.) suggests that the value of the existing 'cohesion' may be the determining factor (the smaller the cover h, the larger the required ground mass cohesion).

C.2.5 Calculation Models

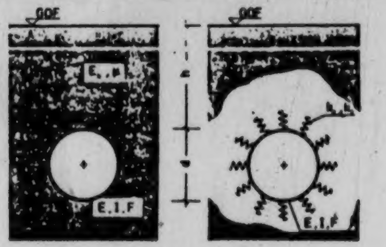
Within the assumption that the design section of the tunnel is situated far enough from the working face area so that plane strain may be assumed, the German recommendations recognize four basic calculation models, depicted in Figure C.3. In the 'semi-continuum model' recommended for shallow tunnels it is important to observe that the full overburden is assumed to be acting at the tunnel crown.

C.3 Design Charts by Ahrens et al. (1982)

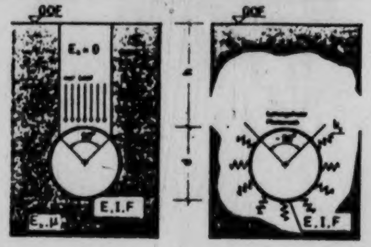
Ahrens et al. (1982) have carried out an extensive investigation of the models presented in Figure C.3. Only the models suggested for shallow tunnels are reviewed herein, mainly because they are presented in the form of simple design charts which were derived from finite element parametric studies.

---

[45] This assumption will normally yield smaller bending moments.

<table>
<tr><th colspan="2">STRUCTURAL DESIGN MODELS RECOGNIZED IN THE GERMAN RECOMMENDATIONS (Duddeck,1980)</th></tr>
<tr><td>DEEP TUNNELS (h > 3d)</td><td>SHALLOW TUNNELS (h < 2d)</td></tr>
<tr><td>

GOF
E_s, μ
E.I.F
h
d
GOF
k_r, k_t
E.I.F

INPUT PARAMETERS REQUIRED:
GROUND: STIFFNESS+POISSON'S RATIO+ DENSITY OR SUBGRADE REACTION MODULI (RADIAL+TANGENTIAL)
SUPPORT: GEOMETRY+YOUNG'S MODULUS</td><td>

GOF
$E_s = 0$
E.I.F
E_s, μ
h
d
GOF
k_r
E.I.F

INPUT PARAMETERS REQUIRED:
GROUND: STIFFNESS+POISSON'S RATIO+ DENSITY OR SUBGRADE REACTION MODULI (RADIAL)
SUPPORT: GEOMETRY+YOUNG'S MODULUS</td></tr>
<tr><td colspan="2">ADDITIONAL REFERENCES:
Ahrens (1976)
Ahrens et al.(1982)
'Taschenbuch fuer den Tunnelbau 1983'</td></tr>
</table>

Figure C.3 Design models recognized in the German recommendations

A brief outline of the assumptions of the model, as well as directives to apply the charts to design are reviewed herein. Mathematical derivations are presented in the paper quoted above and in the publication by Ahrens (1976).

C.3.1 Basic Assumptions

The following assumptions represent the minimum necessary in order to properly understand the design charts:

1. Ground and lining behave linearly elastically.
2. A condition of plane-strain is assumed.

C.3.2 Material Properties

The modulus of the ground to be used in design is termed E_k and is given by:

$$E_k=E_s(1+\nu)(1-2\nu)/(1-\nu) \tag{C.1}$$

where E_s is the constrained modulus. The support forces calculated will be recognizedly dependent on the choice of Poisson's ratio ν. The adoption of values of $\nu \leq 0.35$ will, however, minimize this problem (Ahrens et al., 1982:304).

C.3.3 Ground Support Interaction

The ground support interaction is represented by the coefficient β, given by:

$$\beta=E_k r^3/EI \tag{C.2}$$

where r is the tunnel radius, E is the Young's modulus of the lining and I is the moment of inertia of the lining per unit length.

C.3.4 Load Coefficients

The coefficients presented herein were derived from the original in–situ stresses expressed in polar coordinates and then expanded into Fourier series. The subscripts r and t correspond to the radial and tangential components respectively. Figure C.4 illustrates the notation followed by Ahrens et al. (op.cit.) and present the load coefficients required in the design charts. Ahrens et al. (1982:263) derive the coefficient p_{bieg} and point out that it is rigorously applicable only for the fully embedded case, yielding however acceptable results for the partially embedded case as well. They also observe that the tangential component (p_{t2}) should not be neglected in the determination of p_{bieg} even if a 'radial bond only' condition is employed. Consideration of an eventual extra surcharge (e.g. due to foundation loads) is taken into account by adding the following factor to the coefficient p_{r0}:

$$p=0.5p_A(1+K_0) \tag{C.3}$$

where p_A is the distributed load equivalent to the surcharge.

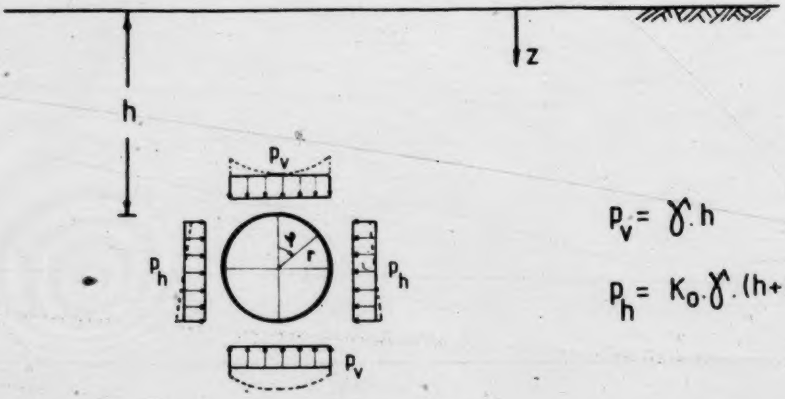
C.3.5 Finite Element Analyses

Ahrens et al.'s (op.cit.) design charts are based on extensive finite element analyses which employ linear elastic isotropic models for the ground and for the lining. The ground is discretized using constant strain triangles and the lining is represented by beam elements. A condition of external loading is applied.

Although the analyses are based on rather simplistic assumptions an effort was made by the authors to evaluate influences like discretization and boundary conditions. Ahrens et al. (1982:265) claim that the boundary conditions chosen do not have any influence on the calculated support forces. It is also interesting to observe that the 'radial bond only' condition was simulated by means of specially developed 'link elements' placed perpendicularly between lining and ground elements. These elements transmit axial loads only and therefore the tangential component is neglected (Ahrens et al., 1982:266).

C.3.6 Design Charts for Shallow Tunnels

The design charts reviewed herein allow the determination of the distribution of bending moments and normal forces in the tunnel lining, as well as the lining deformation at crown. The original notation is kept herein as explained at the bottom of Figures C.5 and C.6. The



$$p_v = \gamma \cdot h$$

$$p_h = K_0 \cdot \gamma \cdot (h+r)$$

$\left.\begin{matrix} p_v \\ p_h \end{matrix}\right] \longleftarrow$ polar coord. + exp. into Fourier series:

$$p_r = \sum^n p_{rn} = \bar{p}_{r0} + \bar{p}_{r1}\cos\varphi + \bar{p}_{r2}\cos 2\varphi + \bar{p}_{r3}\cos 3\varphi, \quad n=0\ldots3$$

$$p_t = \sum^n p_{tn} = \bar{p}_{t1}\sin\varphi + \bar{p}_{t2}\sin 2\varphi + \bar{p}_{t3}\sin 3\varphi \quad n=1\ldots3$$

to use charts:

$\bar{p}_{r0} = 0.5\,\gamma\,(h+r)(1+K_0)$

$\bar{p}_{r2} = 0.5\,\gamma\,(h+r)(1-K_0)$

$\bar{p}_{t2} = 0.5\,\gamma\,(h+r)(1-K_0)$

$p_{bieg} = 2\bar{p}_{r2} + \bar{p}_{t2}$

(for derivation see Ahrens et. al., 1982)

Figure C.4 Load coefficients by Ahrens et al. (1982)

sketch was made by the Author according to a personal communication of Prof. H. Duddeck.

The sequence of calculations is the following:

1. Establish geometry (i.e., tunnel radius r and overburden h) and material properties (ground deformation modulus E_k, Young's modulus E and moment of inertia I for the lining).
2. Calculate the coefficient β according to equation C.2.
3. Calculate the load coefficients p_{r0}, p_{r2}, p_{t2}, and p_{bieg}.
4. Enter charts "A" to determine the bending moment distribution (Figure C.5).
5. Enter charts "B" to determine thrusts and lining deflection at crown (Figure C.6).

Steps 4 and 5 are easily carried out, the correspondent formulas being presented on the charts. The whole process is straightforward and its incorporation in the 'Pocket book for tunnelling construction – 1983' is clearly aimed at providing practitioners with a simple design tool. The charts have been published fairly recently and proper evaluation of their usefulness will require a certain period of time. To the Author's knowledge, no extensive sensitivity studies with the charts have been reported in the literature.

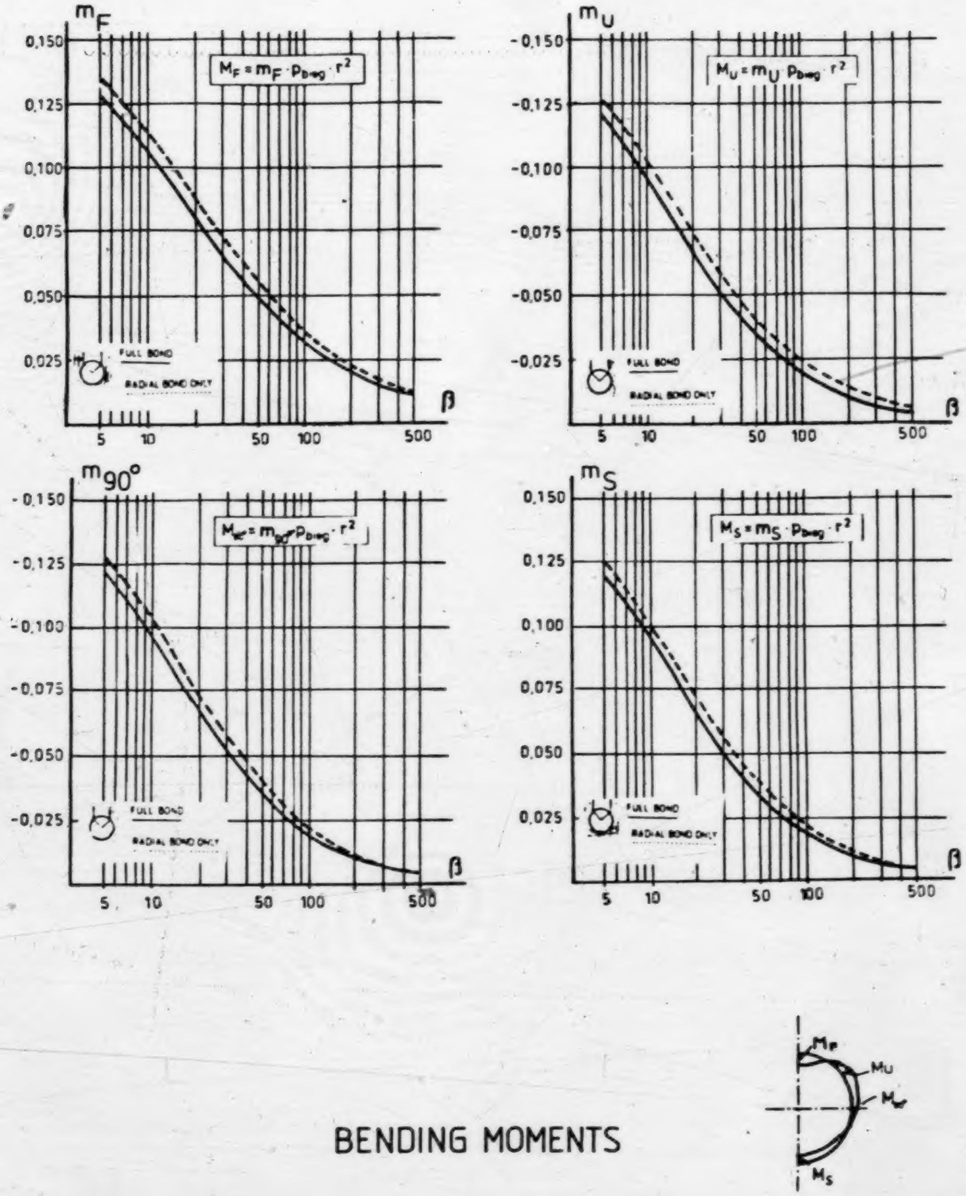
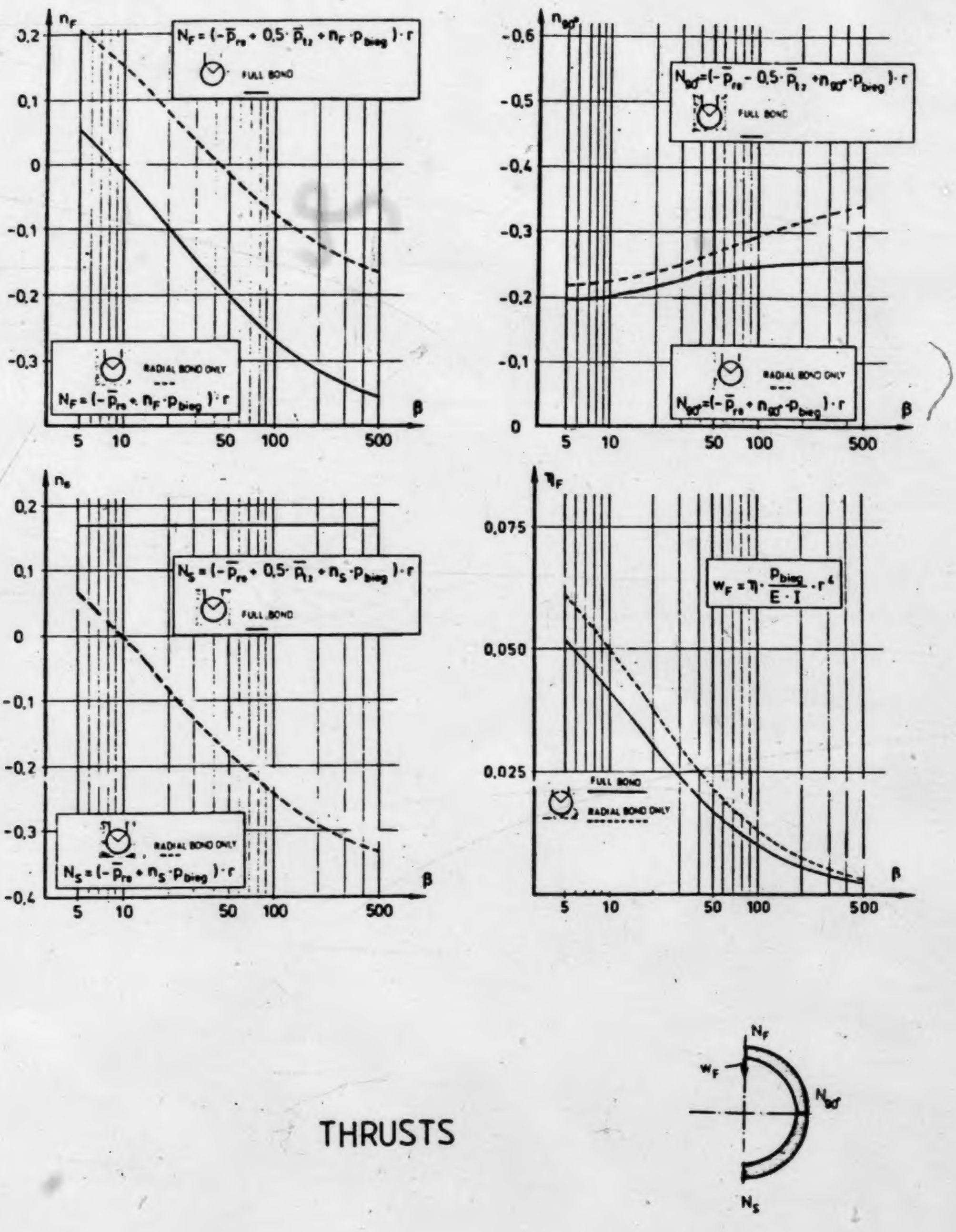


Figure C.5 Charts 'A': Bending moments; modified after Ahrens et al. (1982)



THRUSTS

Figure C.6 Charts 'B': Thrusts; modified after Ahrens et al. (1982)

APPENDIX D - ADINA

Information regarding the use of ADINA were provided in Chapter 4. This appendix contains additional information considered useful for the analyses of tunnels and excavations.

D.1 Excavation Simulation

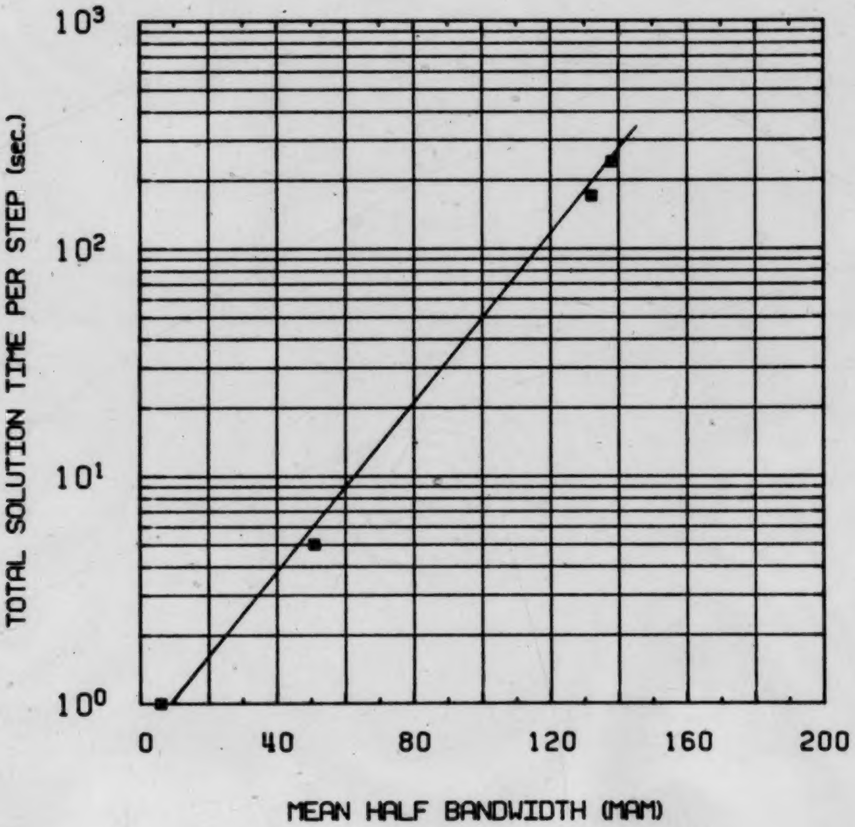
The birth-death option as it was applied to excavation simulation and lining erection is explained in Chapter 4. Additional useful information is provided in the 'ADINA System verification Manual' (ADINA Engng. Report AE 83-5), which recently became available at the Department of Civil Engineering of the University of Alberta.

A remark should be made about the use of beam elements, used to simulate the lining in the analyses of large cross section tunnels reported in Chapter 4. The use of beam and shell elements in problems of soil-structure interaction is frequently suggested in geotechnical literature. Some authors however, point out that problems of compatibility at common nodes may exist when these elements are used in connection with other solid elements (e.g. Naylor and Pande, 1981). Although no problems were verified in the present work, it is suggested that this aspect should be analysed closely in future studies.

D.2 Costs

ADINA is an expensive program to run, especially in the case of 3D non-linear analyses. The chief costs in the 3D analyses reported herein were found to be related to updating stiffness matrices and load vectors for non-linearities and the solution of the large set of equations. In excavation problems, the stiffness has to be reformed at each excavation step, even when linearly elastic materials are used.

A way to obtain a rough evaluation of the costs beforehand is to run a DATACHECK (relatively unexpensive) and obtain the value of MAM—Mean Half Bandwidth, which can be correlated with the total solution time. This is shown in Figure D.1.



ADINA Cost Information (3D Lin. Elastic)

Figure D.1 ADINA Cost Information

APPENDIX E - DERIVATIONS

This appendix contains derivations of ground and support response curves used in Chapter 5.

Derivation of Ground Response Curves:

Elastic GRC equation:

K=1 → Kaiser (1982):

$u/u_o=1-p_i/p_o \therefore (u/a)(a/u_o)=1-p_i/p_o$

Elastic 'hole-in-a-plate' displacements:

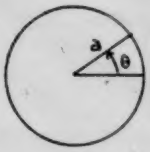
K=1 → $u_o=p_o a/2G$, where $G=E/2(1+\nu)$

K≠1 → Pender (1980, Eqn. 28):

$u_i=a(1+\nu)P_o/2E\{(1+K)-(3-4\nu)(1-K)\cos 2\theta\}$

or:

$u_i=aP_o/4G\{(1+K)-(3-4\nu)(1-K)\cos 2\theta\}$



$\therefore a/u_i=4G/P_o\{(1+K)-(3-4\nu)(1-K)\cos 2\theta\}$

∴ a general equation for the elastic GRC is:

$(u/a)\psi=1-p_i/p_o$ .... (D.1)

where:

$\psi=4G/P_o\{(1+K)-(3-4\nu)(1-K)\cos 2\theta\}$

Check:

For K=1 → $\psi=4G/2P_o$ subst. in eqn. D.1 → $u/u_o=1-p_i/p_o$

Derivation of Support Reaction Lines:

Input:

thickness of support	0.13m
E modulus of support	5000MPa
Poisson's ratio of support	0.25
E modulus of soil	13.7MPa
Poisson's ratio of soil	0.43
tunnel radius (R)	1.95m

$\therefore A_s = 0.13 m^2$ and $I_s = 1.41 \times 10^{-3} m^4$ (per lineal metre)

Entering expressions by Einstein and Schwartz (1979) yields:

$C^* = 4.36$

$F^* = 16.57$

$a_0^* = 0.66$

$\beta = 0.48$

$a_2^* = -0.27$

$b_2^* = -0.55$

Eqn. 19a from Einstein and Schwartz (op.cit.):

$$u_r E/PR(1+\nu) = 0.5(1+K)a_0^* + 0.5(1+K)\{4(1-\nu)b_2^* - 2a_2^*\}\cos 2\theta$$

u_r (mm)	K=0.75	K=1.00
crown	22.39	22.17
springl.	16.40	22.17
invert	22.39	22.17

APPENDIX F - 3D STRESS PATHS

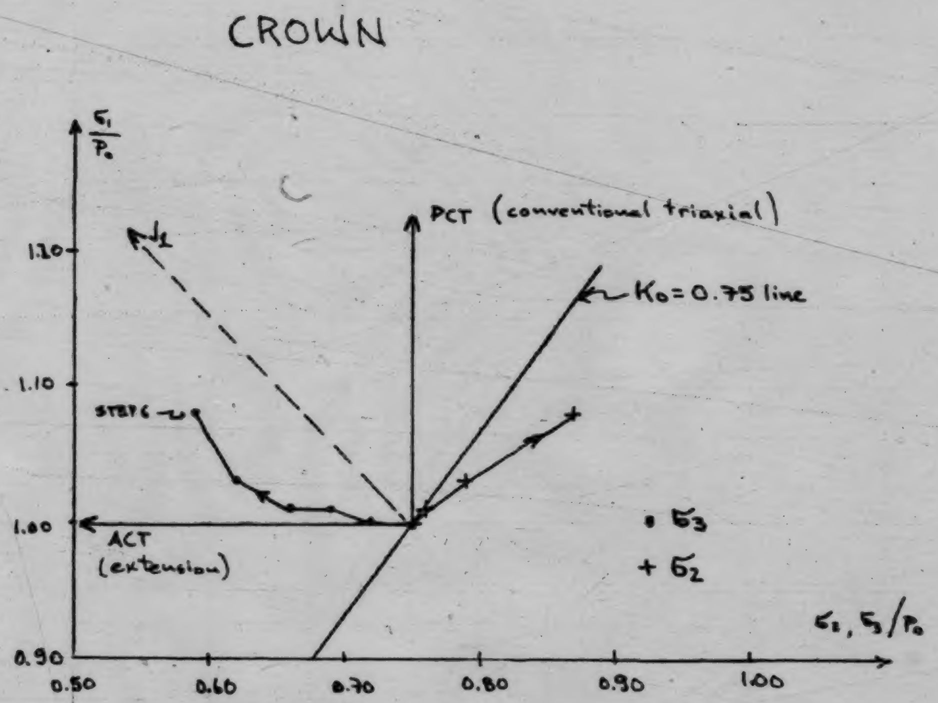
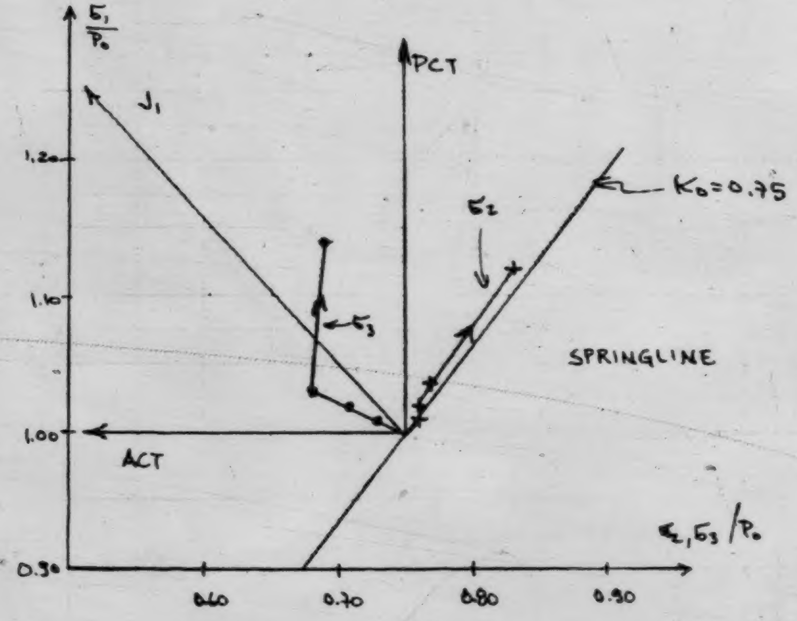


Figure F.1 Stress paths ahead of the face at the crown



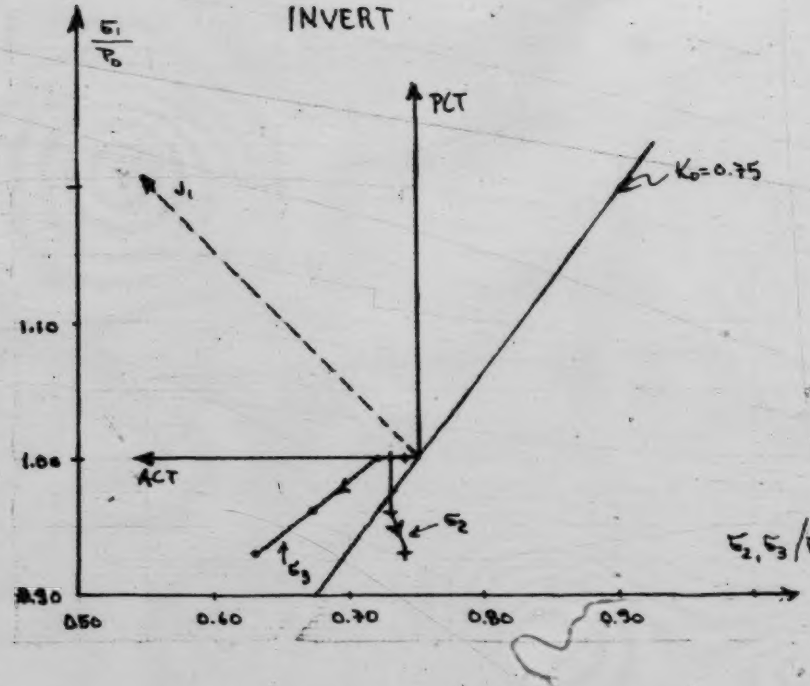


Figure F.2 Stress paths ahead of the face at springline and invert